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19.1 INTRODUCTION

This definition of prestressed concrete as given by the ACI Committee on Prestressed Concrete is:

> "Concrete in which there has been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree. In reinforced concrete members the prestress is commonly introduced by tensioning the steel reinforcement".

This "internal" stress is induced into the member by either of the following prestressing methods:

(1) Pretensioning

In pretensioning the tendons (wires, bars or strands) are first stressed to a given level and then the concrete is cast around them.

The most common system of pretensioning is the "long line" system, by which a number of units are produced at once. First, the wires are stretched between anchorage blocks at opposite ends of the long "stretching bed". Next the spacers or separators are placed at the desired member intervals and then the concrete is placed within these intervals. When the concrete has attained a sufficient strength, the steel is released and its stress is transferred to the concrete via bond.

(2) Post-tensioning

In post-tensioning the concrete member is first cast with a tube or duct for future insertion of tendons. Once the concrete is sufficiently strong, the tendons are stressed by jacking against the concrete. When the desired prestress level is reached the tendons are locked under stress by means of end anchorages or clamps. Subsequently, the duct is filled with grout to protect the steel from corrosion and give the added safeguard of bond.

In contrast to pretensioning, which is usually incorporated in precasting (casting away from final position), post-tensioning lends itself to cast-in-place construction.

19.2 BASIC PRINCIPLES

This section defines the internal stress that results from either prestressing method.

First consider the simple beam shown in Figure 19.2.1.

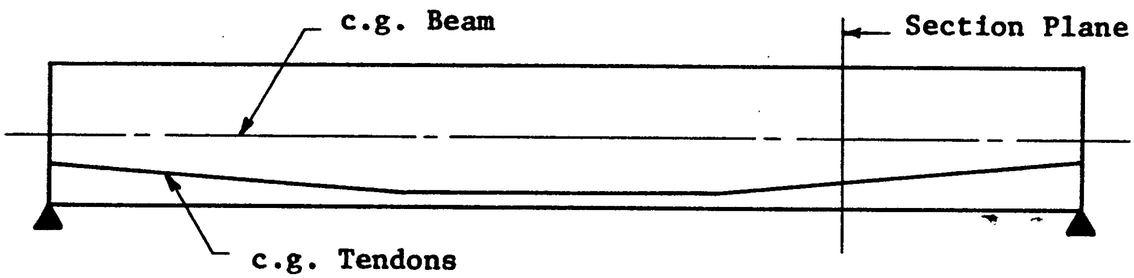


FIGURE 19.2.1

The horizontal component (P) of the tendon force (F) is assumed constant at any section along the length of the beam.

Also, at any section of the beam the forces in the beam and in the tendon are in equilibrium. Forces and moments may be equated at any section.

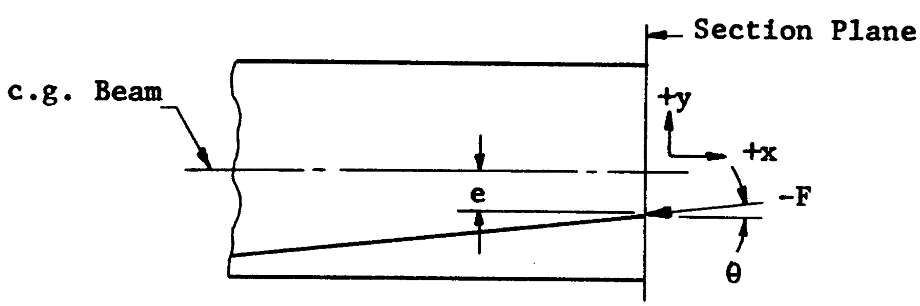


FIGURE 19.2.2

The assumed sign convention is as shown in Figure 19.2.2 with the origin at the intersection of the section plane and the center of gravity (centroidal axis) of the beam. This convention indicates compression as negative and tension as positive.

The eccentricity of the tendon can be either positive or negative with respect to the c.g. therefore it is unsigned in the general equation. Whereas the reaction of the tendon on the beam is always negative, therefore the horizontal component is signed as:

$$-P = -F\cos\Theta \qquad (19.2.1)$$

Then, by equating forces in the 'x' direction the reaction (-P) of the tendon on the concrete produces a (compressive) stress equal to:

$$f_1 = \frac{-P}{A} \qquad (19.2.2)$$

where (A) is defined as the cross-sectional area of the beam.

Since the line of action of the reaction (-P) is eccentric to the centroidal axis of the beam, by the amount (e), it produces a bending moment.

$$M = -Pe \qquad (19.2.3)$$

This moment induces stresses in the beam given by the flexure formula:

$$f_2 = \frac{My}{I} = \frac{-Pey}{I} \qquad (19.2.4)$$

where (I) is the moment of inertia of the section about its centroidal axis and (y) is the distance, unsigned in the general equations, from this axis to the fiber under consideration.

The algebraic sum of equations (19.2.2) and (19.2.4) yields an expression for the total prestress on the section when the beam is not loaded.

$$f_p = f_1 + f_2 = \frac{-P}{A} - \frac{Pey}{I} \qquad (19.2.5)$$

Now, by substituting $I = Ar^2$, where (r) is the radius of gyration, into the above expression and arranging terms we have:

$$f_p = \frac{-P}{A}\left(1 + \frac{ey}{r^2}\right) \qquad (19.2.6)$$

These stress conditions are shown in Figure 19.2.3.

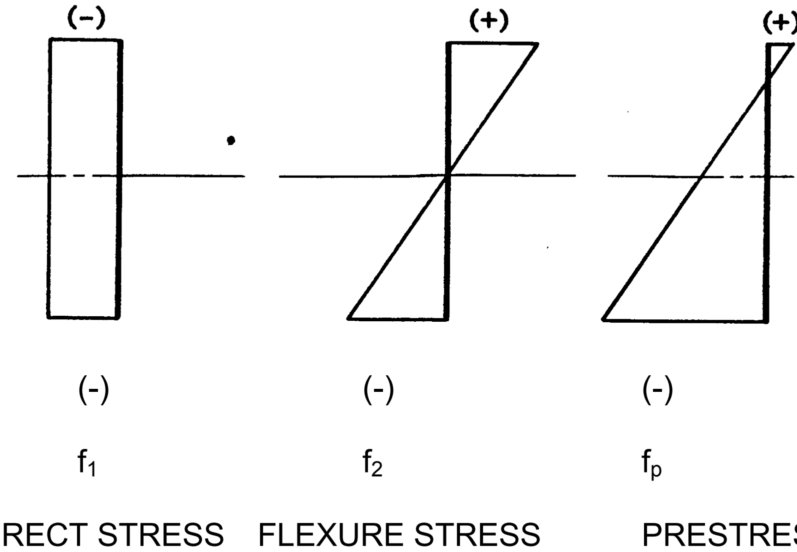


FIGURE 19.2.3

Finally, we equate forces in the 'y' direction which yields a shear force (V) over the section of the beam due to the component of the tendon reaction.

$$V = -F\sin\Theta = -P\tan\Theta \qquad (19.2.7)$$

19.3 PRETENSIONED MEMBER DESIGN

This section outlines the aspects associated with the design of conventional Pretensioned members.

(1) Allowable Stresses

The allowable stresses at different loading stages are defined in Section 9 of AASHTO. The basic ultimate stress for the different structural components are:

Prestressed "I" Girder Concrete	6000 to 8000 psi (42 MPa – 55 MPa)
Slab and Box Type Girder Concrete	5000 psi (35 MPa)
Deck and Diaphragm Concrete	4000 psi (28 MPa)
Prestressed Reinforcement Strand	270 ksi (1860 MPa)
Grade 60 Reinforcement - Yield	60 ksi (420 MPa)

The required concrete strength at time of prestress transfer (fci) is stated on the plans. For Prestressed "I" Girders (fci) min. is 4.8 ksi and for Slab and Box Type Girders (fci) min. is 4.0 ksi. The required concrete strength @ release (fci) is not related to the 28 day girder strength and the amount of curing time required to reach (fci) is not appreciably affected by the required 28 day girder strength.

The use of 8 ksi (55 MPa) concrete for "I" girders and 6.4 ksi for transfer strength (fci) still allows the fabricator to use a 24 hour cycle for girder fabrication. There are situations where higher strength concrete in the "I" girders should be considered for economy. Higher strength concrete should be considered if a line of girders can be saved, if longer spans will eliminate a pier, or to provide a shallower girder.

Prestressed "I" girders below the required 28 day concrete (or 56 day for f'c = 8 ksi) strength will be accepted if they provide strength greater than required by the design and at the reduction in pay schedule in the Wisconsin Standard Specifications for Road and Bridge Construction.

2) Loading Stages

The loads that a member is subjected to during its design life and those stages that generally influence a design are discussed below:

A. Prestress Transfer

This is the initial condition of prestress that exists immediately following the release of the tendons (transfer of the tendon force to the concrete). The eccentricity of the prestress force produces an upward camber in the member which causes the member to span its full length; thereby, inducing a stress due to the dead load of the member itself. This is a stage of temporary stress that includes a reduction in prestress due to elastic shortening of the member.

B. Losses

After losses the "external" loading is the same as at prestress transfer; however, the "internal" stress (prestress) is further reduced by losses resulting from relaxation (creep) of the prestressing steel together with creep and shrinkage of the concrete. The assumption is that all losses occur prior to application of service loading.

Losses that are considered are:

Shrinkage - Loss from concrete shrinkage is 6 ksi (42 MPa). This is based on an average ambient relative humidity in Wisconsin of 72%.

| Elastic Shortening - Elastic shortening loss equals E_s/E_{cn} times f_{cir}.

| Es = 28,000,000 psi
| $E_{ci} = 33 w^{3/2} \sqrt{f'ci}$

| where w is the concrete unit weight in pounds per cubic foot (150) and f'ci is
| the girder concrete strength at the time of initial pressure. For low relaxation
| strands f_{cir} equals the concrete stress at the center of gravity of the
| prestressing steel at the .5 point when the prestressing steel is stressed to
| 0.69 times its ultimate. 0.69 is used for low relaxation strand and 0.63 for
| stress relieved. (f_{cir} is the stress due to the prestressing force and the dead load of beam and may be computed in lieu of using the previous values).

Creep of Concrete - Loss due to creep of concrete equals 12. times f_{cir} - 7. times f_{cds}. f_{cds} equals the concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied. The value f_{cds} is calculated at the .5 point of the span.

Relaxation of Prestressing Steel:

For Low Relaxation Strand

CRS =(5,000-0.10ES-0.05(SH + CRC) - English; units of p.s.i.
= 35 - 0.10ES - 0.05 (SH + CRC) - Metric

where

CRS = loss due to creep of steel psi (MPa)
SH = loss due to concrete shrinkage psi (MPa)
ES = loss due to elastic shortening psi (MPa)
CRC = loss due to creep of concrete psi (MPa)

Fabrication Losses - These losses are not considered by the designer but they affect the design criteria because they exist. Anchorage losses which occur during stressing and seating of the prestressed strands vary between 1 and 4 percent. Losses due to temperature change in the strands during cold weather prestressing are 6% for a 60°F (33°C.) change. The construction specifications permit a 5% difference in the jack pressure and elongation measurement without any adjustment.

AASHTO Specifications permit a temporary strand stress of 85 percent of yield stress with the stress at transfer to be 75 percent for Low Relaxation strands. The fabrication losses account for most or all of this temporary overstress. Therefore, the elastic shortening losses cannot be applied to this temporary condition because of those fabrication losses.

C. Service Load

Here the member is subjected to the same loads as after prestress transfer and losses occur along with the effect of either of the following:

1. In the case of the "I" girder type the dead load of the deck and diaphragm are always carried by the basic girder section on a simple span basis. This is due to the type of construction used; i.e., nonshored girders simply spanning from one substructure unit to another for single as well as multi-span structures.

   The live load plus impact along with any post-dead load (curb, parapet or median strip which is placed after the deck concrete has hardened) are carried by the composite section.

   In the case of multi-span structures the longitudinal distribution for the live, impact and post-dead loads are based on a continuous span structure. This continuity is achieved by:

   (a) Placing non-prestressed (conventional) reinforcement in the deck area over the interior supports.

   (b) Casting concrete between and around the abutting ends of adjacent girders to form a diaphragm at the support.

   Live load continuity is not considered for girders resting on expansion bearings where the diaphragm does not bear on the top of the pier unless a positive moment connection is made between the girders. If the span length ratio of two adjacent spans exceeds 1.5, the girders are designed as simple spans. In either case, the stirrup spacing is detailed the same as for continuous spans and bar steel is placed over the supports equivalent to continuous span design. This value of 1.5 is not a structural limit as the positive moment reduces as this ratio increases. However, the reduction in strands for the reduced moment may not offset the increased girder cost in the shorter spans if the girders are not removed. For simple spans, the girder spacing can be increased in the end spans.

2. In the case of the slab and box type girders with a bituminous or thin concrete surface the dead load together with the live load plus impact are carried by the basic girder section.

   Whereas, when this type has a concrete floor the dead load of the floor is carried by the basic section and the live load, impact, and any post-dead loads are carried by the composite section. A composite floor of 3" (75 mm) minimum thickness is recommended.

   Note that the slab and box type girders are generally used for single span structures. Therefore, both dead and live loads are carried on a simple span basis.

   Occasionally, these girders are used on continuous spans with a concrete floor. Transverse cracking is likely to occur at piers with a shallow deck. To control this cracking, detail a moveable joint at the piers or consider using a composite floor thickness of 5" (130 mm). A single mat of bar steel or welded wire fabric is recommended for composite section construction. Reference is made to Standard 19.15 for Pretensioned Slab and Box Section details.

D. Ultimate Load

At this stage, the ultimate strength of the composite section is considered, except for the bituminous floor or thin concrete surface case where the basic section applies. Since the member is designed on a service load basis it must be checked for its overload ability at the ultimate stress condition.

The need for both service load and ultimate strength computations lies with the radical change in a member's behavior when cracks form. Prior to cracking the gross area of the member is effective. As a crack develops, all the tension in the concrete is picked up by the reinforcement. If the percentage of reinforcement is small, there is very little added capacity between cracking and failure.

(3) Design Procedure

The intent is to provide the designer with a general outline of steps for the design of pretensioned members.

A. Transverse Member Spacing

A trial 'I' girder arrangement is made by using Table 19.1 as a guide. An ideal spacing results in equal strands for interior and exterior girders together with an optimum slab thickness. The final prestressed girder design should have a minimum Operating Rating of HS30 (MS27). Current practice is to use a minimum haunch of 2" (50 mm) for design calculations, and then use a 2 1/2" (60 mm) average haunch for concrete quantity calculations as outlined in Section U - Deck Forming. The maximum practical slab overhang is discussed in Chapter 17 of this manual.

The pretensioned slab or box is used in a "multi-beam" system only; i.e., precast units are placed side by side and locked (post-tensioned) together. The span length, desired roadway width and live loading control the size of the member.

* (3' (915 mm) wide section vs. 4' (1220 mm) wide section): Do not mix 3' (915 mm) wide and 4' wide sections across the width of the bridge. Examine the roadway width produced by using all 3' sections or all 4' sections and choose the system that is the closest to but greater than the required roadway width. For a given section depth and desired roadway width, a multi-beam system with 4' sections can span greater lengths than with 3' sections. Therefore if 3' sections were the best choice for meeting roadway width criteria; and section depth can't be increased, but span length was too long for this system, then examine switching to all 4' sections to meet this required span length. Tables stating the approximate span limitations as a function of section depth and roadway width are on Tables 19.2 and 19.3, respectively.

B. Live Load Distribution

This criteria is found in AASHTO Section 3, Part C - "Distribution of Loads".

For exterior 'I' type girders the distribution factor (DF) for moment equals the reaction of a unit wheel load obtained by assuming the deck to act as a simple span between girders.

For interior 'I' type girders the $DF = \frac{S}{5.5}$ where (S) is the girder spacing.

The distribution of loads for determining end reactions is defined in Bridge Manual Chapter 27 - Bearings.

With the 'multi-beam' system refer to AASHTO 3.23.4 for the fraction of wheel load, S/D applied to interior members.

C. Live Load and Impact

The impact formula is stated in AASHTO. It is applicable to all superstructure members.

The live load moments and shears can be obtained from Computer Programs.

The exterior pretensioned slab or box in a 'multi-beam' system is designed as a longitudinal 'edge beam' capable of resisting a live load moment LLM = 0.10 P x SPAN, where P is a wheel load.

D. Deck Design

The deck thickness and required top and bottom mat reinforcement is a function of the effective span and live load as shown in Bridge Manual Chapter 17 – Superstructure General.

E. Dead Load

The dead load moments and shears are computed for simple spans. When post-dead loads are considered; these post-dead load moments are based on continuous spans for concrete deck slabs.

A post-dead load of 20#/ft^2 (1 kN/m^2) is to be included in all designs which accounts for a possible future concrete overlay wearing surface.

The curb or parapet dead load is distributed equally to all girders for design.

F. Composite Section

| A portion of the deck width is taken as a composite top flange. For <u>interior</u> "I"
| girders the effective width of the composite flange is taken as the least of the following:

1. One-fourth the span of the girder.

2. The center to center distance of girders.

3. Twelve times the effective deck thickness plus the width of the girder <u>web</u>.

| For the 54W, 72W or 82W girders an effective web width of 34.25 inches may
| be used as stated in AASHTO 9.8.3.

The effective deck thickness equals the overall thickness minus the 1/2" (15 mm) concrete wearing surface.

For <u>exterior</u> "I" girders the effective flange width is taken as the full overhang from the C/L of girder <u>plus</u> the least of the following:

1. One-eighth the span of the girder. (Due to overhang meeting criteria for top flange on both sides of web).

2. Six times the effective deck thickness plus 1/2 the girder web.

3. One-half the distance to the next girder.

The composite flange area for an interior "multi-beam" is taken as the width of the member by the effective thickness of the floor. Minimum concrete overlay thickness is 3" (75mm).

The composite flange for the exterior member consists of the curb and the floor over that particular "edge beam".

Since the deck concrete has a lower strength than the girder concrete it has a lower modulus of elasticity. Therefore, when computing composite section properties the effective flange width as stated above must be reduced by the ratio of the modulus of elasticity of the deck concrete divided by the modulus of elasticity of the girder concrete. For 6000 psi girder concrete E is 5,500,000 psi and for 4,000 psi deck concrete E is 4,125,000 psi. For this combination the ratio (4125/5500) is equal to 0.75. For girder concrete strengths other than 6,000 psi, E is calculated from the following formula:

$$E = 5{,}500{,}000 \; x \sqrt{f'c} \; / \; \sqrt{6{,}000} \; (psi)$$

For slab concrete strength other than 4,000 psi, E is calculated from the following formula:

$$E = 4{,}125{,}000 \; x \sqrt{f'c} \; / \; \sqrt{4{,}000} \; (psi)$$

The E value of 5,500,000 psi for 6,000 psi girder concrete strength is determined from non-composite dead load deflections measured in the field.

G. Design Stress

The design stress is the bottom fiber stress due to dead load on the basic girder section together with live load, impact and post-dead load on the composite section.

The point of maximum stress is generally 0.5 of the span for both end and intermediate spans. But for longer spans (100' (30 meters) or so) the 0.4 point of the end span may control and is to be checked.

H. Prestress Force

With the design stress known, compute the required prestress needed to counteract all the design stress except that residual amount of tension which is allowed $(0.498\sqrt{f'_c})$. [English $6.0\sqrt{f'_c}$].

The required prestress (f_p) equals the design stress minus the allowable residual stress.

For slab type girders assume a strand pattern eccentricity (e) and apply this to Equation 19.2.6 to determine the final prestress force (P) and the approximate number of strands. Then a trial strand pattern is established using Standard 19.15 as a guide and a check is made on the assumed eccentricity.

Whereas for "I" type girders, Equation 19.2.6 is solved for several predetermined patterns and is tabulated in the Standards.

Present practice is to detail all spans of equal length with the same number of strands, unless a span requires more than 3 additional strands. In this case the different strand arrangements are detailed along with a plan note stating: "The manufacturer may furnish all girders with the greater number of strands".

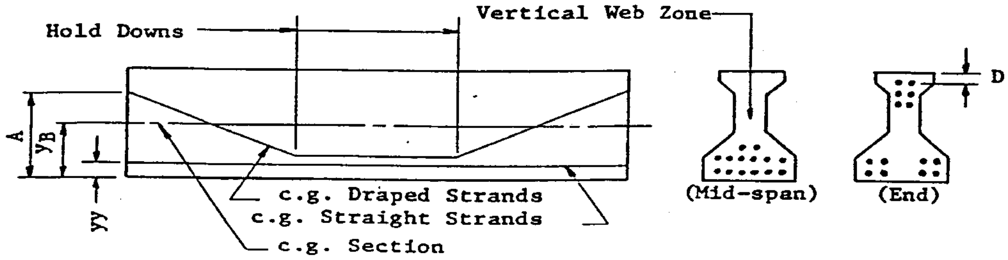
I. Service

Check top and bottom girder stresses along with stress at top of composite slab at the design point of the span. This confirms compliance with the allowable stress criteria for top of girder and top of slab since only the bottom stress is used to determine the prestress. Maximum allowable concrete compressive stresses after losses are: slab-1.6 ksi (11 MPa) and girder = 0.6 f'c except: (1) 0.4 f'c due to effective prestress and permanent dead loads; and (2) 0.4 f'c for live load stresses and 1/2 the stresses in case (1).

J. Strand Drape

This consists of draping some of the strands in order to decrease stresses due to prestress at the ends of the 'I' girder where the stress due to loads are minimum.

The typical strand profile for this technique is shown below.



Note that all the strands that lie within the 'vertical web zone' of the mid-span arrangement are used in the draped group.

The allowable initial tension and compression stresses are given in AASHTO. These stresses are a function of f'ci, the compressive strength of concrete at the time of initial prestress.

Holddown points for draped strands are located at or between the 1/3 point and the 4/10 point from each end of the girder. Standards 19.1 thru 19.5, 19.18 and 19.21 show "B" values at the 1/4 point of the girder. The minimum "B" value shown will locate the holddown at the 1/3 point. The maximum "B" value ($B_{MIN.}$ + 3" (75 mm)) should be checked and reduced if necessary to keep the holddowns within the limits specified above.

The maximum slope specified for draped strands is 12%. This limit is determined from the safe uplift load per strand of commercially available strand restraining devices used for holddowns. The minimum distance "D" allowed from center of strands to top of flange is 2" (50 mm). Initial girder stresses are checked 2'-6” (760 mm) from the girder end because this is the approximate length of embedment required to fully develop the strand stress. For most designs the maximum allowable slope of 12% will determine the location of the draped strands. Using a maximum slope will also have a positive impact on shear forces.

An alternate to draping is that of <u>raised</u> strand patterns for 'I' type girders. Present practice is to show a standard raised arrangement as an alternate to draping for <u>short</u> spans. For longer spans bond breakers at the ends of the strands is an alternate (the limit is 25% of the strands). Show only one strand size for the draped pattern on the plans. Use only 0.5” strand for the draped pattern on the 28 and 36” girders. Use 0.6” strand for the straight pattern and all patterns on the other girders, but drape no more than 6 strands on the 45”

The strands in slab and box type girders are normally not draped, but instead are arranged to satisfy the stress requirements at mid-span and ends of girder.

K. Strength at Transfer

The required strength at transfer (f'_{ci}), which is stated on the plans, is computed based on the temporary compressive stress due to initial prestress and girder dead load.

$$f_c = f_{DL} + F_P \leq (0.6 f_{ci})$$

The controlling bottom fiber stress occurs at either the holddown or girder ends of draped designs and at girder ends of non-draped designs. For f'_c = 6 ksi (42 MPa), maximum allowable girder stress is equal to 0.6(5200)= 3120 psi (21.6 MPa).

L. After Losses

Check the bottom fiber stress at:

1. The above control point due to girder dead load and effective prestress.

2. The continuous end of the girder due to negative live load moment and effective prestress.

Note that the stress in 1 need not be checked at the simple end of draped designs since the drape criteria covers this.

Generally the stresses in 1 and 2 meet the "design stress" criteria.

M. Non-prestressed Reinforcement

This consists of bar steel reinforcement used in the conventional manner. It is placed longitudinally along the top of the member to carry any tension which may develop after transfer of prestress. The designer should completely detail all rebar layouts including stirrups.

The amount of reinforcement is that which is sufficient to resist the total tension force in the concrete based on the assumption of an uncracked section.

For draped designs the control is at the holddown point of the girder. Here initial prestress together with girder dead load stress is acting. This is where tension due to prestress is still maximum and compression due to girder dead load is decreasing.

For non-draped designs the control is at the end of the member where prestress tension exists but dead load stress does not.

Note that a minimum amount of reinforcement is specified in the Standards. This is intended to help prevent serious damage due to unforeseeable causes like improper handling or storing.

N. Ultimate Capacity

The required ultimate positive moment capacity is expressed as multiples of dead load (DL), live load (LL), and impact (I).

$$M_u = 1.3(DL + 5/3(LL + I))$$

These load factors are applicable for both simple and continuous span structures.

When sidewalk L.L. is present, $M_u = 1.3(DL + 1.25((LL + I) + SDWLL))$.
The equations for determining the ultimate flexural strength for a girder with a composite concrete deck are stated in AASHTO Specifications. Use a capacity reduction factor "ϕ" equal to 0.95 for flexure computations.

O. Steel Percentage

The maximum and minimum steel percentage criteria is given in AASHTO and it applies to both girder types. However, this requirement need not be checked for 'I' girders since the pre-selected strand pattern is based on this criteria.

P. Horizontal Shear

This is the stress between the member and its composite deck at ultimate shear.

$$V_u = 1.3(DL + 5/3(LL + I))$$

$$\bar{V}_u = V_u / \phi$$

Use a capacity reduction factor "ϕ" equal to 0.85 for shear.

The dead load in this case is only that which acts on the composite section.

The shear stress between the contact surface is

$$v = \frac{\bar{V}_u Q}{Ib}.$$

where:

Q = statical moment of the effective transferred area of the deck taken about the centroidal axis of the composite section.

I = moment of inertia of the composite section.

b = width of the contact area between the member and the composite deck.

Laboratory investigations[1, 2] indicates that the horizontal shear stresses permitted by AASHTO are very conservative. It is on the basis of these laboratory tests that the following criteria is based:

The allowable horizontal shear stress is 300 psi (2.0 MPa)when the contact surfaces are clean and intentionally roughed. When the horizontal design shear is greater refer to AASHTO.

This criteria is applied at the support having maximum vertical shear. The stirrups on the Standards are considered adequate for slabs with composite concrete floors.

Q. Web Reinforcement

This consists of placing conventional reinforcement perpendicular to the axis of the "I" type girder. The required area of web reinforcement is:

$$A_v = \frac{(V_u - V_c)S}{2f_{sy}jd} \quad \text{(or } \frac{50b's}{f_{sy}} \text{ min)}$$

(b') the web thickness, (j) may be taken as 0.88 and (S) is the spacing of the reinforcement, which is limited to 3/4 of the girder depth (or 24 inches). Use a capacity reduction factor "ϕ" equal to 0.90 for shear.

$$V_u = 1.3(DL + 5/3(LL + I)); V_u = V_u / \phi$$

Determine V_c as the minimum of either V_{ci} or V_{cw} where

$$V_{cw} = (3.5\sqrt{f'c} + .3fpc)b'd \quad \text{(English)}$$

$$V_{ci} = .6\sqrt{f'c}b'd + V_d + \frac{V_i Mcr}{M_{\max}} \quad \text{(English)}$$

$$V_{cw} = 10x10^5[.29\sqrt{f'c} + .3f_{pc})b'd] \quad \text{(Metric)}$$

$$V_{ci} = 4.98x10^4\sqrt{f'_c}b'd + V_d + \frac{V_j M_{cr}}{M_{\max}} \quad \text{(Metric)}$$

Assume a composite section. V_{ci} corresponds to shear at locations of accompanying flexural stress. V_{cw} corresponds to shear at simple supports and points of contraflexure. Critical computation for V_{cw} is at the centroid for composite girders.

See AASHTO "Notations" for an explanation of the preceding terms in V_{cw} and V_{ci}.

Value (DL) is the dead load of the girder, deck and post-dead loads when applicable.

Note that the vertical component of the draped strands which helps to reduce the shear on the concrete section is neglected for conservatism.

Reinforcement, in the form of vertical stirrups, is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at a unit stress of 20 ksi (138 MPa) and are placed within d/4 of the girder end. This is a conservative design but is currently employed for crack control.

Welded wire fabric may be used for the vertical members. It must be deformed wire with a minimum size of D16.

R. Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on ultimate strength methods.

$$M_u = 1.3(DL + 5/3(LL + I))$$

Use the appropriate "ø" factors

$$V_u = 1.3(DL + 5/3(LL + I))$$

AASHTO Specifications allow a capacity reduction factor equal to 1.0 for factory produced precast prestressed concrete members.

Here the dead load is only that which is placed after the monolithic support diaphragm and deck have set up.

The amount of reinforcement is based on Load Factor Design as shown in Chapter 18 of this Manual. Refer to the Table in Chapter 17 for the percentage of bar steel required based on the value of $Mu/ ø\, bd^2$.

The concrete between the abutting girder ends is usually of a much lesser strength than that of the girders. But, tests[4] have shown that due to lateral confinement of the diaphragm concrete the girder itself fails in ultimate negative compression rather than the material between its ends. Therefore the ultimate compression stress (f'_c) of the girder concrete is used in place of that of the diaphragm concrete.

This assumption has only a slight effect on the computed amount of reinforcement, but it has a significant effect on keeping the compression force within the bottom flange.

The transverse spacing of the continuity reinforcement is usually taken as the whole or fractional spacing of the "D" bars as given in Chapter 17. Grade 60 (Grade 420) bar steel is used for continuity reinforcement.

Required development lengths for deformed bars are given in Bridge Manual, Chapter 9. Current practice is to carry bars past the cutoff point a distance equal to the girder depth for development length requirements.

The length of continuity reinforcement is based on the following criteria:

1. When one-half the bars satisfy the moment diagram, terminate one-half the bars.

2. When the top fiber stress of the girder equals the modulus of rupture, terminate the remaining one-half of the bars. This point is to be at least 1/20 of the span length or 4' (1200 mm) from point 1, whichever is greater.

Certain secondary features result when spans are made continuous.

That is, positive moments develop over piers due to creep[5] together with those produced by live plus impact loads in remote spans. The latter only exists for bridges with three or more spans.

These positive moments are somewhat counteracted by negative moments resulting from differential shrinkage[6] between the cast-in-place deck and precast girders along with those due to post-dead loads.

At the present time it is not considered necessary to provide a positive moment connection at the pier if the bearing is fixed and the diaphragm is full depth and anchored to the pier; this is based primarily on past experience.

S. Precompression

The precompression in the bottom flange, at the girder end due to prestress, is neglected in the ultimate moment computation if the following limitations are met:

1. The maximum precompression stress is less than 0.4 f'_c. This is always met because of the drape criteria.

2. The percentage of continuity reinforcement is less than 1.5%[4,5].

T. Camber and Deflection

The prestress camber and dead load deflection are used to establish vertical position of the deck forms with respect to the girder.

Figure 19.3.6 typifies a girder with a draped strand profile.

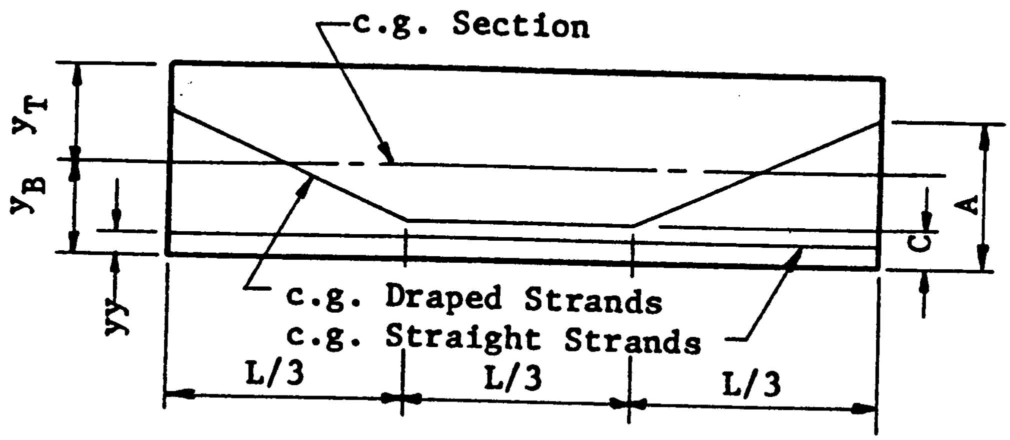


FIGURE 19.3.6

1. Prestress Camber

The eccentric straight strands induce a constant moment of:

$$M_1 = P_i^s(y_B - yy)$$

where (P_i^s) is the initial prestress force in the straight strands minus the elastic shortening loss. This moment produces an upward deflection at mid-span.

$$\Delta_s = \frac{M_1 L^2}{8EI}$$

The draped strands induce moments at the ends and within the span:

$M_2 = P_i^D$ (A-C) produces upward deflection

$M_3 = P_i^D$ (A-y_B) downward when (A) is greater than (y_B)

Where (P_i^D) is the initial prestress force in the draped strands minus the elastic shortening loss. These moments produce a net upward deflection at mid-span.

$$\Delta_D = \frac{L^2}{8EI} \quad (\frac{23}{27} M_2 - M_3)$$

The combined upward deflection due to prestress is:

$$\Delta \Pr estress = \Delta_s + \Delta_D$$

$$= \frac{L^2}{8EI} (M_1 + \frac{23}{27} M_2 - M_3)$$

The downward deflection due to dead load is:

$$\Delta = \frac{5WL^4}{384EI}$$

Where (W) is the member weight per unit length, (I) is the moment of inertia and (E) the modulus of elasticity which is assumed at 3.5 x 10^6 psi (24000 MPa), for f'c = 6 ksi concrete, for these deflections at time of release. See Section 19.3(F) for (E) when f'c > 6 ksi. Therefore, the anticipated prestress camber is the net prestress deflection minus the members dead load deflection.

2. Dead Load Deflection

The downward deflection due to the dead load of the deck and mid-span diaphragm is:

$$\Delta_{DL} = \frac{5WL^4}{384EI} + \frac{PL^3}{48EI}$$

where (W) is the deck weight per unit length and (P) the weight of the diaphragm, both based on that carried by a typical interior girder. The modulus of elasticity (E) is assumed at 5.5 x 10^6 psi (38,000 MPa), for f'c = 6 ksi concrete, for mature concrete and (I) is the moment of inertia of the basic member.

3. Residual Camber

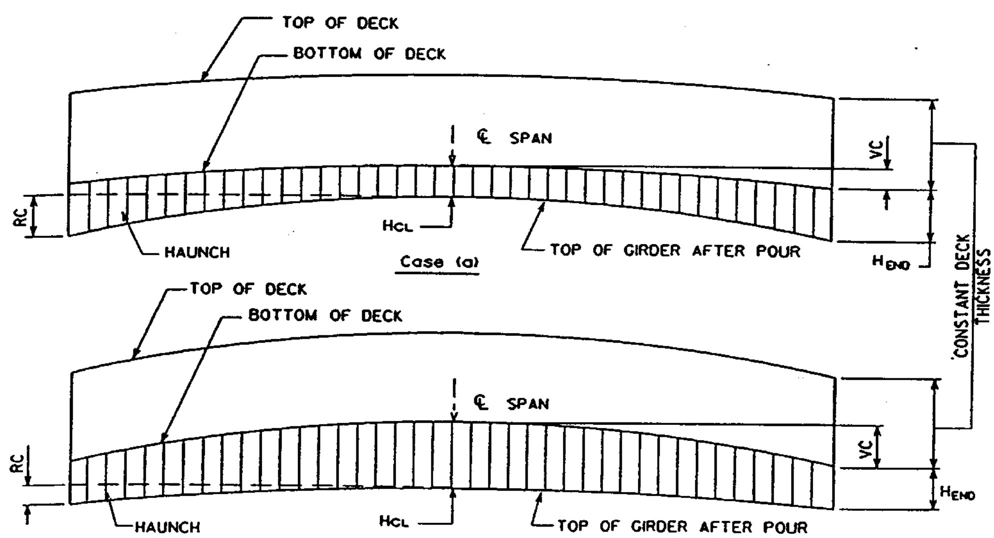
This is the camber that remains after the prestress camber has been reduced by the dead load deflection.

U. Deck Forming

This consists of computing the relationship between the top of girder and bottom of deck necessary to achieve the desired vertical roadway alignment.

Present practice is to use a minimum haunch of 2" (50 mm) for design and average of 2 1/2" (60 mm) for computing the haunch concrete quantity. This will facilitate current deck forming practices which use 1/2" (13 mm) removable hangers and 3/4" (19 mm) plywood, plus allow for variations in prestress camber. Also, future deck removal will be less likely to damage the top girder flanges.

For designs involving vertical curves, Figure 19.3.7 is given showing two cases.



Case (b)

FIGURE 19.3.7

In case (a) the vertical curve amount (VC) is less than the computed residual Camber (RC) and therefore the minimum haunch occurs at mid-span. Whereas in case (b), the (VC) is greater than (RC), where the minimum haunch occurs at the girder ends.

For equal span continuous structures, having all spans on the same vertical alignment, the deck forming is the same for each span. This is due to the constant change of slope of a vertical curve or tangent and the same (RC) per span.

The following equation is derived from Figure 19.3.7.

END MID-SPAN

| $$(+H_{end}) = (RC) - (VC) + (+H_{ci})$$

where upward (RC) is positive and (VC) is positive for crest curves and negative for sag curves.

For unequal spans or when some spans are on the curve and others on the tangent a different approach is used. Generally the longer span or the one off the curve dictates the haunch required at the common support. Therefore, it is necessary to pivot the girder about its mid-span in order to achieve an equal condition at the common support. This is done Mathematically by adding together the equation for each end (abutment and pier).

LEFT END RIGHT END MID-SPAN

| $$(+H_{lftend}) + (+H_{rtend}) = 2[(RC) - (VC) + (+H_{ci})]$$

with the condition at one end known, due to the adjacent span the condition at the other end is computed.

V. Construction Joints

Locate the transverse construction joints in deck midway between the cut-off points of the continuity reinforcement or at the .75 point of the span, whichever is closest to the pier. Locate joint at least 1' (300 mm) from cut-off points.

This change keeps stresses in the slab steel from slab dead load at a minimum and makes deflections from slab dead load closer to the theoretical.

X. Strand Types

| Currently low relaxation strand (sizes 0.5” and 0.6”) is used in prestressed

concrete "I" girder designs and shown on the plans. Strand patterns and initial
| prestressing forces are given on the plans and deflection data is shown. If
| requested by the girder fabricator, 0.5" special strand can be furnished.

Y. Construction Dimensional Tolerances

Refer to the Standard Specifications for Road and Bridge Construction for the required dimensional tolerances.

Z. Prestressed Girder Sections

Prestressed girder sections used by the Bridge Office vary depending on vertical clearance requirements and structure span lengths. The 36 and 45 (915 and 1145) "I" shapes are standard AASHTO sections. The 70" (1780) "I" shape and has been used as an economic section on longer spans in Wisconsin since 1971. These sections employ draped strand patterns with undraped alternates where feasible. The 54W" "I" shape was developed in 1999 to replace the standard AASHTO 54" section.

The 28" (710) prestressed "I" section was developed in 1985 as a viable alternate for bridge spans in the 30' to 50' (9 to 16 meter) range. Undraped strand patterns when practical should be specified on the designs. For these sections, the cost of draping far exceeds savings in strands. For the 16 and 18 strand patterns, bond breakers are required. The bond breaker is composed of a plastic sleeve which fits over the strand. This isolates the strand from the surrounding prestressed girder concrete. Refer to Standard 19.9 for strands requiring bond breakers.

Bond breakers should only be applied to interior strands as girder cracking has occurred when they were applied to exterior strands. In computing bond breaker lengths, consideration is given to the theoretical stresses at the ends of the girder. These stresses are due entirely to prestress; as a result the designer may compute a stress reduction based on certain strands having bond breakers. This reduction can be applied along the length of the debonded strands. Reference is made to the fact that the prestressing strands have development lengths of approximately 30" (760 mm).

Table 19.1 a,b provides span lengths versus interior girder spacings for HS20 (MS18) live loading on single span & multiple span structures for prestressed "I" girder sections. Girder spacings are based on using low relaxation strands at 0.75 f's, a concrete haunch of 2" (50 mm), slab thicknesses from Table 17.1, future wearing surface, and assumes no parapet or sidewalk load distributed to the interior girders.

Tables 19.2 and 19.3 provide approximate span limitations for "pretensioned slab and box sections" as a function of section depth and roadway width. They also give the section properties associated with these members. Criteria for developing these tables is shown below Tables 19.2 and 19.3. The Tables are in English Units.

AA. Precast, Prestressed Slab or Box Sections

These Sections may be used with skews greater than 30° up to an absolute maximum skew of 45°. The transverse post tensioning is placed along the skew.

When these Sections are in contact with water for 5-year flood events or less, the Sections must be cast solid for long term durability. When these Sections are in contact with water for the 100-year flood event or less, any voids in the Section must be cast with a non water absorbing material.

TABLE 19.1a MAXIMUM SPAN LENGTH VS. GIRDER SPACING FOR INTERIOR PRESTRESSED CONCRETE “I” GIRDERS, 0.6” DIAM. STRANDS, f’c GIRDER = 8,000 psi, f’c SLAB = 4,000 psi, HAUNCH HEIGHT = 2” REQUIRED f’c GIRDER @ INITIAL PRESTRESS < 6400 psi

28” GIRDER

Girder Spacing	Single Span	2 Equal Spans
6’-0	57’	61’
6’-6	55’	59’
7’-0	53’	57’
7’-6	52’	55’
8’-0	50’	54’
8’-6	48’	52’
9’-0	47’	50’
9’-6	44’	49’
10’-0	42’	47’
10’-6	41’	44’
11’-0	39’	43’
11’-6	38’	42’
12’-0	37’	40’

36” GIRDER

Girder Spacing	Single Span	2 Equal Spans
6’-0	73’	78’
6’-6	70’	75’
7’-0	68’	73’
7’-6	66’	70’
8’-0	64’	68’
8’-6	62’	66’
9’-0	55’	64’
9’-6	54’	62’
10’-0	52’	56’
10’-6	51’	54’
11’-0	49’	53’
11’-6	48’	51’
12’-0	47’	50’

45” GIRDER

Girder Spacing	Single Span	2 Equal Spans
6’-0	102’	108’
6’-6	97’	105’
7’-0	94’	102’
7’-6	91’	97’
8’-0	89’	95’
8’-6	85’	92’
9’-0	82’	90’
9’-6	80’	85’
10’-0	78’	83’
10’-6	74’	80’
11’-0	68’	78’
11’-6	67’	75’
12’-0	65’	71’

54W" GIRDER

Girder Spacing	Single Span	2 Equal Spans
6'-0	134'	138'
6'-6	130'	136'
7'-0	126'	133'
7'-6	123'	130'
8'-0	120'	127'
8'-6	117'	124'
9'-0	114'	121'
9'-6	111'	118'
10'-0	106'	114'
10'-6	104'	112'
11'-0	101'	107'
11'-6	99'	105'
12'-0	95'	103'

72W" GIRDER

Girder Spacing	Single Span	2 Equal Spans
6'-0	*161'	*165'
6'-6	*157'	*164'
7'-0	153'	*161'
7'-6	149'	*157'
8'-0	146'	*154'
8'-6	143'	150'
9'-0	139'	147'
9'-6	136'	144'
10'-0	132'	139'
10'-6	129'	136'
11'-0	125'	132'
11'-6	123'	130'
12'-0	120'	127'

| * For lateral stability during lifting these girder lengths will require pick up point locations greater than distance "d" (girder depth) from the ends of the girder. The designer shall assume that the pick up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange if required.

| Due to the wide flanges on the 54W and 72W and the variability of residual camber, haunch heights frequently exceed 2". Do not push the span limits during preliminary design.

TABLE 19.1b MAXIMUM SPAN LENGTH VS. GIRDER SPACING FOR INTERIOR PRESTRESSED CONCRETE “I” GIRDERS, 0.6” DIAM. STRANDS, f’c GIRDER = 8,000 psi, f’c SLAB = 4,000 psi, HAUNCH HEIGHT = 2”, REQUIRED f’c GIRDER at INITIAL PRESTRESS < 8,000 psi

28” GIRDER

Girder Spacing	Single Span	2 Equal Spans
6’-0	61’	65’
6’-6	59’	63’
7’-0	57’	61’
7’-6	55’	59’
8’-0	53’	57’
8’-6	51’	55’
9’-0	50’	54’
9’-6	48’	52’
10’-0	47’	50’
10’-6	46’	49’
11’-0	43’	47’
11’-6	42’	46’
12’-0	41’	44’

36” GIRDER

Girder Spacing	Single Span	2 Equal Spans
6’-0	78’	83’
6’-6	75’	80’
7’-0	73’	78’
7’-6	69’	75’
8’-0	66’	73’
8’-6	64’	70’
9’-0	60’	66’
9’-6	56’	64’
10’-0	52’	60’
10’-6	51’	57’
11’-0	49’	53’
11’-6	48’	51’
12’-0	47’	50’

45” GIRDER

Girder Spacing	Single Span	2 Equal Spans
6’-0	106’	108’
6’-6	103’	108’
7’-0	100’	105’
7’-6	96’	102’
8’-0	94’	100’
8’-6	91’	97’
9’-0	88’	94’
9’-6	85’	91’
10’-0	82’	89’
10’-6	78’	86’
11’-0	73’	83’
11’-6	70’	80’
12’-0	65’	76’

54W" GIRDER

Girder Spacing	Single Span	2 Equal Spans
6'-0	138'	142'
6'-6	135'	140'
7'-0	132'	138'
7'-6	128'	136'
8'-0	125'	132'
8'-6	122'	129'
9'-0	119'	126'
9'-6	116'	123'
10'-0	112'	119'
10'-6	109'	117'
11'-0	106'	113'
11'-6	104'	111'
12'-0	101'	108'

72W" GIRDER

Girder Spacing	Single Span	2 Equal Spans
6'-0	*163'	*169'
6'-6	*160'	*166'
7'-0	*157'	*164'
7'-6	*154'	*161'
8'-0	150'	*158'
8'-6	147'	*154'
9'-0	144'	151'
9'-6	140'	148'
10'-0	136'	143'
10'-6	133'	140'
11'-0	129'	136'
11'-6	126'	133'
12'-0	124'	131'

* See Note on Page 29.

TABLE 19.2
(4′-0 SECTION WIDTH)
SECTION PROPERTIES---AND---MAXIMUM SPAN LENGTH

MAXIMUM RECOMMENDED SPAN LENGTH, (FT).
VALUES IN ( ) = S/D---AASHTO 3.23.4.3
VALUES IN $\boxed{}$ = # PRESTRESSED STRANDS PRESENT

Section No.	Section Depth	Section Area (in²)	Moment of Inertia (in⁴)	Section Modulus (in³)	24′ Rdwy. Width (6 girder sys.)	26′ Rdwy. Width (7 girder sys.)	28′ Rdwy. Width (7 girder. sys.)	30′ Rdwy. Width (8 girder sys.)
1	1′-0	576	6912	1152	23′ $\boxed{9}$ (.694)	23′ $\boxed{9}$ (.701)	23′ $\boxed{9}$ (.701)	23′ $\boxed{9}$ (.709)
2	1′-5	555	18606	2189	38′ $\boxed{14}$ (.688)	38′ $\boxed{14}$ (.695)	38′ $\boxed{14}$ (.695)	38′ $\boxed{14}$ (.701)
3	1′-9	595	32942	3137	48′ $\boxed{16}$ (.681)	48′ $\boxed{16}$ (.686)	48′ $\boxed{16}$ (.686)	48′ $\boxed{16}$ (.691)
4	2′-3	668	64187	4755	61′ $\boxed{18}$ (.674)	61′ $\boxed{18}$ (.678)	61′ $\boxed{18}$ (.678)	61′ $\boxed{18}$ (.682)
5	2′-9	728	107204	6497	72′ $\boxed{19}$ (.671)	72′ $\boxed{19}$ (.674)	72′ $\boxed{19}$ (.674)	72′ $\boxed{19}$ (.677)
6	3′-6	818	196637	9364	86′ $\boxed{20}$ (.667)	86′ $\boxed{20}$ (.670)	86′ $\boxed{20}$ (.670)	86′ $\boxed{20}$ (.673)

Table based on... HS20 Live Load (2 traffic lanes)
... All girder sections (4′-0 width)
... Simple span structure
... f′c = 5,000 p.s.i. (girder section); max. fci = 4,500 p.s.i.
... 1/2" ϕ-prestressing strands @.75 f′s (low relaxation); f′s = 270.0 k.s.i.
... 2" min. concrete overlay (which doesn't contribute to stiffness of section)
... includes 20#/ft² (f.w.s) load
... assumed "F"-rail wgt. distrib. evenly to all girder sections
... in AASHTO 3.23.4.3; k = 0.7 (sect. #1) and k = 1.0 (sect. # 2-6)
... max. # of prestr. strands were used w/o exceeding allow. stress limits @ ends
... span length is from CL-CL brg.

TABLE 19.3
(3′-0 SECTION WIDTH)
SECTION PROPERTIES---AND---MAXIMUM SPAN LENGTH

Section No.	Section Depth	Section Area (in²)	Moment of Inertia (in⁴)	Section Modulus (in³)	MAXIMUM RECOMMENDED SPAN LENGTH, (FT.) VALUES IN ( ) = S/D---AASHTO 3.23.4.3. VALUES IN $\boxed{}$ = # PRESTRESSED STRANDS PRESENT			
					24′ Rdwy. Width (8 girder sys.)	26′ Rdwy. Width (9 girder sys.)	28′ Rdwy. Width (10 girder sys.)	30′Rdwy. Width (10 girder sys.)
1	1′-0	432	5184	864	23′ $\boxed{7}$ (.521)	23′ $\boxed{7}$ (.525)	23′ $\boxed{7}$ (.529)	23′ $\boxed{7}$ (.529)
2	1′-5	435	14036	1651	37′ $\boxed{10}$ (.517)	37′ $\boxed{10}$ (.521)	37′ $\boxed{10}$ (.525)	37′ $\boxed{10}$ (.525)
3	1′-9	475	25012	2382	45′ $\boxed{11}$ (.512)	45′ $\boxed{11}$ (.515)	45′ $\boxed{11}$ (.518)	45′ $\boxed{11}$ (.518)
4	2′-3	548	49417	3660	56′ $\boxed{12}$ (.507)	56′ $\boxed{12}$ (.510)	56′ $\boxed{12}$ (.512)	56′ $\boxed{12}$ (.512)
5	2′-9	608	83434	5057	66′ $\boxed{13}$ (.504)	66′ $\boxed{13}$ (.507)	66′ $\boxed{13}$ (.509)	66′ $\boxed{13}$ (.509)
6	3′-6	698	155317	7396	79′ $\boxed{14}$ (.502)	79′ $\boxed{14}$ (.503)	79′ $\boxed{14}$ (.505)	79′ $\boxed{14}$ (.505)

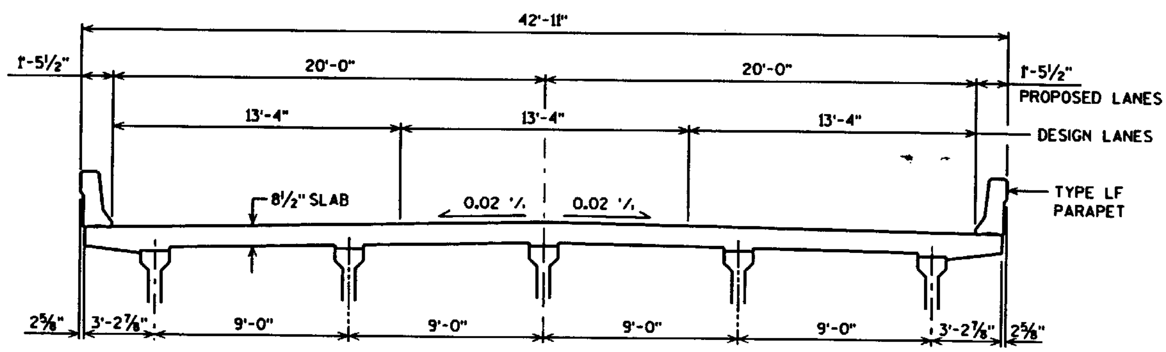
Table based on... HS20 Live Load (2 traffic lanes)
... All girder sections (3′-0 width)
... Simple span structure
... f'c = 5,000 p.s.i. (girder section); max. fci = 4,500 p.s.i.
... 1/2" ϕ - prestressing strands @ .75 f's (low relaxation); f's = 270.0 k.s.i.
... 2" min. concrete overlay (which doesn't contribute to stiffness of section)
... includes 20#/ft². (f.w.s.) load
... assumed "F"-rail wgt. distrib. evenly to all girder sections
... in AASHTO 3.23.4.3; k = 0.7 (sect. #1) and k = 1.0 (sect. # 2-6)
... max. # of prest. strands were used w/o exceeding allow. stress limits @ ends
... span length is from CL - CL brg.

19.4 DESIGN EXAMPLES

A. I Type Girder

This is an example of a three span continuous structure. The same basic procedure is applicable to single spans except live load continuity reinforcement computations are not required.

Trial Roadway Cross Section

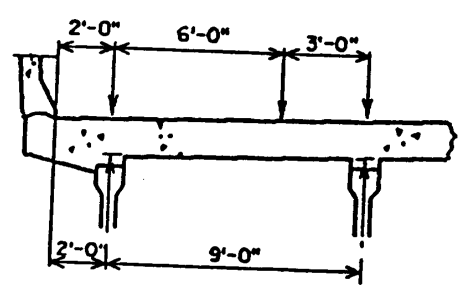


Design Data TRY 5-45" PRETENSIONED GIRDERS
HS20-CONTINUOUS FOR LIVELOAD
L.L. DESIGN SPANS: 80'-1 1/2, 80'-9, 80'-1 1/2
D.L. DESIGN SPANS: 79'-6, 79'-6, 79'-6 (c/c BRG)
OVERALL GIRDER LENGTH: 80'-6 (Use 1 Diaphragm)
1/2" f LOW RELAXATION STRANDS
20 #/Ft. FUTURE WEARING SURFACE
f'_c GIRDER = 6.0 Ksi
f'_c SLAB = 4.0 Ksi

LL Distribution

Interior DF = 9.0/5.5 = 1.64 wheels/girder

Exterior DF = (3.0 + 9.0)/9.0 = 1.33



Max. LL + Impact Moments & DL Moments at Pier

Values are from the "Continuous Beam Analysis: - Prestressed "I".

	Span 1 & 3 (POS)	PIER (NEG)	Span 2 (POS)
Int. LLM+I	= 11351 in-k	9115 in-k	9368 in-k
Ext. LLM+I	= 9205 in-k	7392 in-k	7597 in-k
Composite DL Mom (Int.)		1393 in-k	
Composite DL Mom (Ext.)		4014 in-k	

Deck Design

Effective span = (9'-0") - (1-4") = 7'-8

Deck T = 8.5" From Table in Chapter 17 (An 8" slab would have worked)

Dead Load

	Interior		Exterior
Deck = .708 x 9.0 x 150	= 956#/'	.708 x 7.58 x 150 + 2.58 x .21 x 150/2	= 845#/'
Haunch = 2 x 16 x 150/144	= 33#/'[1]		= 33#/'[1]
Girder	= 583#/'		= 583#/'
Diaphragm (MC 18 x 42.7)*(9-.58) + 45 (Connection Angles)	= 405#	405/2	= 203#

DL Moments - Span 1

Deck + Haunch DLM = $(0.989)(80.125)^2(12/8)$

= 9524 in-k $(0.878)(80.125)^2(12/8)$ = 8455 in-k

Girder DLM = $(.583)(80.125)^2(12/8)$ = 5614 = 5614

Diaph. DLM = (0.41)(80.125)(12/4) = 98 = 49

DLM = 15236 in-k 14118 in-k

Composite Dead Load

Future Wearing Surface = 9.0 x 20 = 180#/' Int., 6.50 x 20 = 130#/' Ext.

Parapet = 387#/' (Assume Carried by Exterior Girder Only)

Span 1 & 3 CDLM: Int = 1108 in-k, Ext. = 3190 in-k

Span 2 CDLM: Int. = 367 in-k, Ext. = 1058 in-k

Composite Section

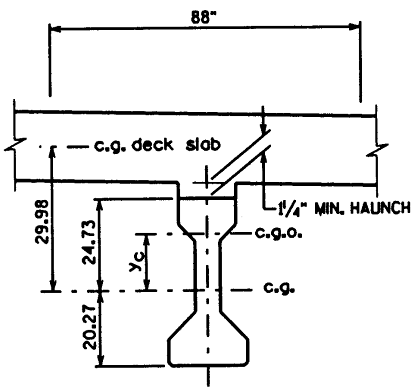
Interior

T_{eff} = 8-1/2" - (1/2") Wearing Surface = 8"; B = (12 x 8) + 7 = 103"

$S_B = 10912\ in^3$ $S_T = 44629\ in^3$

Exterior

T_{eff} = 8"; B = 37 + 6 x 8 + 3 = 88"



Transformed Area = 88 x 8 x .75 = 528 in^2

Tran. $I_o = 88 \times (8)^3 \times .75/12 = 2816\ in^4$

Sum Moments About c.g.

Item	A	Y	Ay	Ay^2	I_o	I_o+Ay^2
Girder	560	0	0	0	125390	125390
Slab	528	29.98	15829	474567	2816	477383
Sum	1088	-	15829	-	-	602773

$-A\ Y_c^2$ - <u>230311</u>

| $Y_c = \sum Ay/\sum A$ = 15829/1088 = 14.55"

I_{comp} = 372462 in^4

Y_t = 24.73 - 14.55 = 10.18" S_T = 372462/10.18 = 36588 in^3

Y_B = 20.27 + 14.55 = 34.82" S_B = 372462/34.82 = 10697 in^3

Design Stress (Bottom mid-span)

		Span 1 & 3	Span 2
Interior f_{DL}	$= \frac{15236}{6186}$	$= 2.463$ ksi	$\frac{15236}{6186} = 2.463$ ksi
f_{LL+I}	$= \frac{11351}{10912}$	$= 1.040$	$\frac{9368}{10912} = 0.859$
f_{CDL}	$= \frac{1108}{10912}$	$= \frac{0.102}{3.605}$	$\frac{367}{10912} = \frac{0.034}{3.356}$
Minus Tension Allowed:	$6\sqrt{f'c}$	$= \frac{-.465}{3.140 \text{ ksi}}$	$= \frac{-.465}{2.891 \text{ ksi}}$
Exterior f_{DL}	$= \frac{14118}{6186}$	$= 2.282$ ksi	$\frac{14118}{6186} = 2.282$ ksi
f_{LL+I}	$= \frac{9205}{10697}$	$= 0.861$	$\frac{7597}{10697} = 0.710$
f_{CDL}	$= \frac{3190}{10697}$	$= \frac{0.298}{3.441 \text{ ksi}}$	$\frac{1058}{10697} = \frac{0.099}{3.091 \text{ ksi}}$
		$\underline{-.465}$	$\underline{-.465}$
		$= 2.976$ ksi	$= 2.626$ ksi

Prestress Force

Assume total losses for 45" Girder = 49 ksi
% Losses = 49/202.5 = 24.2% Say 24%
Estimated initial stress = Design Stress/(1-% Losses)
= 3.140/0.76 = 4.132 ksi (Int. controls Span 1 & 3)

Try 30 Strands (See f_B (Init.) on Standard 19.11)

(Estimate the Elastic Shortening Concrete Loss as 7%)

Initial Force = 0.68 x 270 x 0.153 x 30 = 843 Kips
f_B = (-843/560)[1 + (-16.13 x -20.27/223.9)] = 3.703 ksi (Bot. Fiber)

f_T = (-1.505)[1 - (16.13 x 24.73/223.9)] = +1.176 ksi (Top Fiber)

Initial Prestress + Beam Dead Load (Mid-Span, Elastic Shortening of Concrete Loss)

f_B = - 3.703 + 5614/6186 = -2.795 ksi

f_T = + 1.177 - 5614/5070 = +0.070 ksi

Elastic Shortening Loss

Stress at c.g. of Strands (5/10 pt)
f_{cir} = -2.795 + [(2.795 + 0.070) (20.27 - 16.13)/45] = -2.613 ksi
ES = 7f_{cir} = 7 x 2.613 = 18.3 ksi, 100 (18.3/270) = 6.8% (Estimated 7%)

Creep of Concrete
Stresses due to Deck DLM and Diaphragm DLM (There is no Curb DLM).
f_B = 9622/6186 = 1.555 f_T = -9622/5070 = 1.898 ksi
Stress at c.g. of Strands (5/10 pt)
f_{cds} = 1.555 - [(1.555 + 1.898) (20.27 - 16.13)/45] = 1.237 ksi
CRC = 12 f_{cir} - 7 f_{cds} = 12 x 2.613 - 7 x 1.237 = 22.7 ksi

Shrinkage = SH = 6 ksi based on average relative humidity of 72%

Creep of Prestressing Steel

CRS = 5. - .10 ES - 0.05 (SH + CRC) = 5. - (.10)(18.3) - 0.05 (6 + 22.7) = 1.7 ksi
Service Load (At 5/10 Point with all Losses)
Losses = 6. + 18.3 + 22.7 + 1.7 = 48.7 ksi

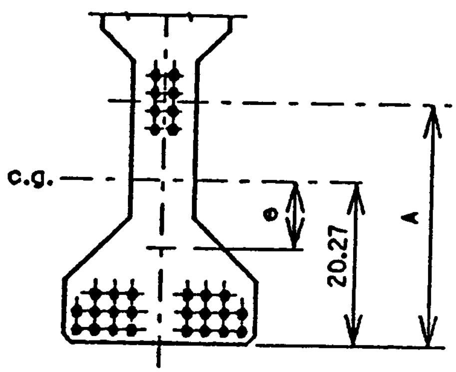
(202.5-48.7)/202.5 = 0.759 say 0.76

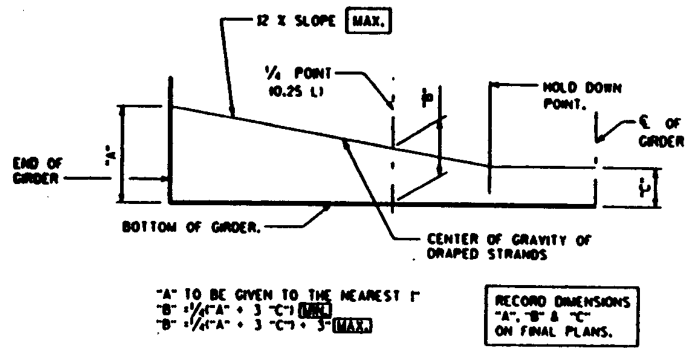
f_B = 4.086 x 0.76 + 3.556 = - 3.105 + 3.556 = + 0.451 < 0.465 OK

Max. Allowable Compression Stress = 0.6 f'c (All Loads)
or 0.4 f'c (LLM + I + 1/2 Dead Load Stresses)

f_T = [(-930/560)(1 + (-16.13 x 24.73/223.9) x 0.76 - (14906/5070) - 1120/43154] * .5
- (11351/43154 = [0.986 - 2.940 - 0.026] * .5 - .263 = - 1.253 < 2.4 ksi OK or
.986 - 2.940 - 0.026 - .263 = 2.243 < 3.6 ksi

Strand Drape - Find c.g. of Draped Strands to meet top and bottom allowable stresses at end.





LOCATION OF DRAPED STRANDS

WISDOT practice is to place upper most draped strands 2" (50 mm) clear from the top of girder or at a maximum slope of 12% in order to reduce shear stresses.

A = 45 - 5 = 40"
B = 13.75" Minimum or B = 16.75" Maximum

Check Max. Slope: (40-13.75)*100/(80.125x12/4)=10.92% < 12% OK

Compute e = (8x40 + 8x2 + 8x4 + 6x6)/30 = 404/30 = 13.47
e = 20.27 - 13.47 = 6.80"

Check Top Fiber Stress Equal to Zero

$f_T = 0 = \frac{-P}{A} (1 - [ey_T/r^2])$, = (-930/560) (1-6.80 x 24.73/223.9) = -413 psi < 0. OK

Check Bottom Fiber Stress Equal to 3.6 ksi After Losses

f_B = 0.76 x (-930/560) (1 + 6.80 x 20.27/223.9) = -2.039 ksi < 3.6 OK

Determine Bottom End Stress Before Losses except ES Loss

f_B = (-843/560) [1 + (-6.80 x -20.27/223.9)] = -2.323 ksi

Strength at Transfer (f'_{ci})

Check at Hold Down (1/3 Point) Interior Girder After ES Loss

Assume Parabolic Distribution of Dead Load
1/3 Point Ordinate = 3.33 x 6.67/25 = 0.89
Girder f_{DL} = 0.89 x 5614/6186 = 0.808 ksi
f_B = -3.703 + 0.808 = 2.895 ksi
f'_{ci} = 2.895/0.6 = 4.825 ksi $\underset{\sim}{<}$ 5.4 ksi OK

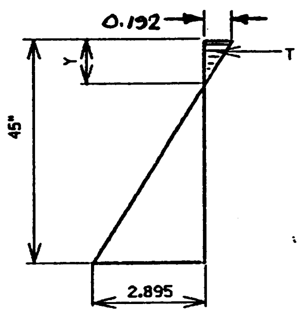
Check at Girder End After ES Loss
f_B = 2.323 ksi OK

Specify: (Plan Values)

30 1/2" Strands with 8 Draped.
A = 40" B = 13.75" Min. or 16.75" Max.
Minimum Prestress Force at Initial Concrete Set is:
0.75 x 270 x 0.153 x 30 = 930 Kips
Maximum Allowable Temporary Prestress Force is:
0.81 x 270 x 0.153 x 30 = 1004 Kips which allows for the inaccuracies and losses associated with stressing the strands.
Minimum Concrete Compressive Strength at time of transfer is 4800 psi.

Non-prestress Reinforcement Required in Top Flange (Hold down point)

$f_T = +1.176 - 0.89x5614/5070 = +0.192 < 7.5\sqrt{4800} = 0.520\ OK$
$f_B = -2.895$ ksi



(Similar Triangles)

Y = (0.192 x 45)/(0.192 + 2.895) = 2.80"

T = 1/2(2.80) x 0.192 x 16 = 4.3 Kips

A_S = 4.3/24 = 0.18 in^2

Ultimate Capacity (Positive)

Interior Girder Span 1 & 3 Controls
Compute at 0.5 point where LLM = 11045 in-k
M_u = 1.3 (15236 + 1108) + 2.17 x 11045 = 45215 in-K

Assume Rectangular Section:

| Cap. $M_u = \phi A_s f_{su} d (1\text{-}0.6 p f_{su}/f_c')$ if $1.4\ d p f_{su}/f_c' < 8"$

Girder depth	= 45.0"
Eff deck	= 8.0
Girder haunch	= 1.25
C.G. strands	= -4.13
	50.12"

$A_s = 30 \times .1531 = 4.6\ in^2$ $p = 4.6/(103 \times 50.12) = .000891$

$f_{su} = f_s' (1\text{-}.5 p f_s'/f_c') = 270 (1 - .5 \times .000891 \times 270/4.0) = 262$ ksi

$1.4 \times 50.12\ (.000891) \dfrac{262}{4.0} = 4.6 < 8"$

Cap. $M_u = .95 \times 4.6 \times 262 \times 50.12 (1 - 0.6 \times .000891 \times 262/4.0) =$ 55377 in-k OK

Cracking Moment (Minimum Steel Percentage)

Prestress After Losses: $f_B = -3.105\,ksi$ $\quad f_R = 7.5\sqrt{6000} = 0.580\,ksi$

Bottom Dead Load Flange Stress after losses:

$f_B = 3.105 - 14906/6186 = 1108/10912 = 0.593$

Live Load Moment which will crack girder:

$M_{LL} = (.580 + .593) \times 10912 = 12800$ in-k

$M_{CR} = 14906 + 1108 + 12800 = 28814$ in-k

Moment Ratio = 55377/28814 = 1.922 > 1.2 OK

Horizontal Shear Between Slab and Girder

Minimum tie reinforcement shall not be less than 50 $b_v S/fy$ where S shall not exceed 4 times least web thickness or 24".

Min. Stirrup Area = 50 x 16 x 24/60000 = 0.32 in^2 @ 24" (2-#4's=0.40 in)

Web Reinforcement

| Int. $V_u = 1.3 \times 5.8 + 2.17 \times 56.8 = 130.8$ K $V_u/\phi = 153.8$K

d = 45.0 +8.0 - 4.63 = 49.62 j = 0.88 jd = 43.67"

$V_c = 180 \times 8 \times 43.67 = 62.9$ Kips

A_v = Pair #4's = 0.4 in^2

S = (0.4 x 60 x 43.67)/(153.8 - 62.9) = 11.5" = #4's @ 10" OK

Continuity Reinforcement

Find Exterior Continuity Reinforcement (Refer to Chapter 17 Table for values of Rho)

M_u = 1.3 (3622 + 5/3 (7392)) = 20750 in-k
d = 45.0 + 8.0 - 2.75 + 1.25 = 51.5 in.
R_u = $M/\phi bd^2$ = 20,750,000/(.9)(22)$(51.5)^2$ = 395 psi
p = 0.0070; A_s = (0.0070)(22)(51.5) = 7.93 in^2 (Controls)

<u>Check Fatigue Requirement</u> (Assume ff = 23 ksi)
A_s = (7392)/(23)(0.9)(51.5) = 6.93 in^2
A_s = 6.93/(4.5+3.23) = 0.90 in^2/ft.

<u>Find Interior Continuity Reinforcement</u>

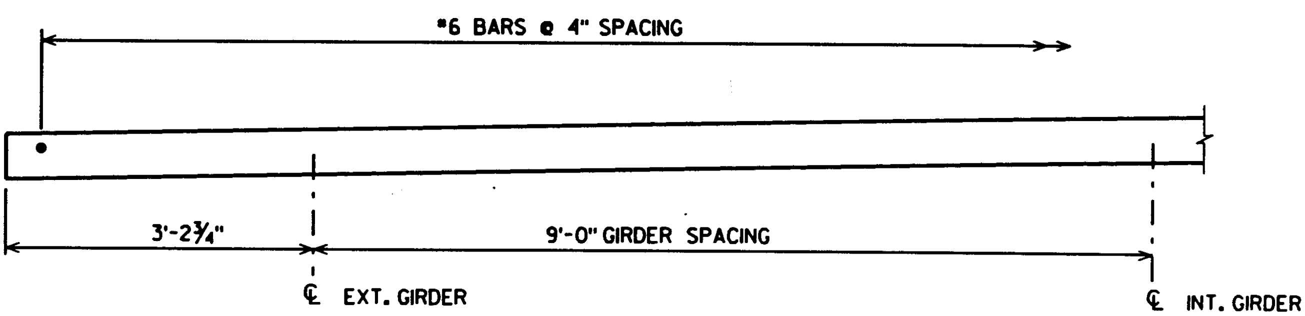
M_u = 1.3 [1393 + 5/3 (9115)] = 21591 in-k
Ru = 21,591,000/.9(22)$(51.5)^2$ = 411 psi
p = 0.0074; As = (0.0074)(22)(51.5) = 8.38 in^2

<u>Check Fatigue Requirement</u>

A_s = (9115)/(23)(0.9)(51.5) = 8.55 in^2 (Controls)
A_s = 8.55/9 = 0.95 in^2/ft.

From Table #5 @ 4' = 0.92 in^2/ft.

Note: The smallest practical bar size is selected providing the most effective distribution of steel. Space bars at multiples of bottom longitudinal bars. Assume maximum size aggregate of 1 1/2". Minimum clear distance between bars = 1.5 x 1.5" = 2.25". The #4 bars at the bottom of the thickened slab may be included as part of the continuity steel.



Check Crack Control

Crack Control allowable stress is compared to fatigue allowable stress since both values are based on live moment alone. Crack control when computed for an interior beam is solely a function of live loading. For all practical purposes, the future wearing surface acts to seal the cracks and is not expected to cause cracking in the same manner as live loading. The 1/2" wearing surface is deducted when computing allowable stress for crack control.

Allowable $f_s = Z/(d_cA_c)^{1/3}$ where Z = 130 and d_c = 2.0" (Max. d_c = 2")

At Pier: A_c = 5 x 5 = 25 in^2 f_s = 130/(2.0 x 25)$^{1/3}$ = 35.3 ksi which is greater than the allowable f_f = 23.0 ksi OK

At First Cutoff: A_c = 5 x 10 = 50 in^2 f_s = 130/(2.5 x 50)$^{1/3}$ = 28.0 ksi

Ultimate Negative Moment Values (Span 1 & 3) From Prestress Program

Span 1 Point	0.6	0.7	0.8	0.9	C/L Pier
2.17 x LLM (Int.)	-574	-668	-765	-962	-1648 ft-k
1.30 x Post DLM	0*	0*	0*	-68*	- 151
	-574	-668	-765	-1030	-1799
2.17 x LLM (Ext.)	-465	-545	-620	-782	-1336
1.30 x Post DLM	+233	+135	-2	-177	-392
	-232	-410	-622	-959	-1728

* Load Factor = 0.975 since moment is of opposite sign. Do not include positive moment of future wearing surface.

Negative Moment Cutoff Points (Spans 1 & 3)

Moment capacity of one-half bar area (22 - #6's = 4.84 sq. in.)

p = A_s/bd = 4.84/(22)(51.5) = 0.00427 Ru = 245.2 psi

M = Ru/ϕbd^2 = 245.2/(.9 x 22 x 51.5^2) = 12,876 in-k/12 = 1073 ft-k

Terminate 1/2 the bars at about the .90 span point plus.

Offset Distance = Eff. Depth Eff. Depth = 4.13'± c.g. Prestressing Steel

(Top fiber = modulus of rupture = 580 psi = $7.5\sqrt{f_c'}$)

0.8 Span e = -13.26

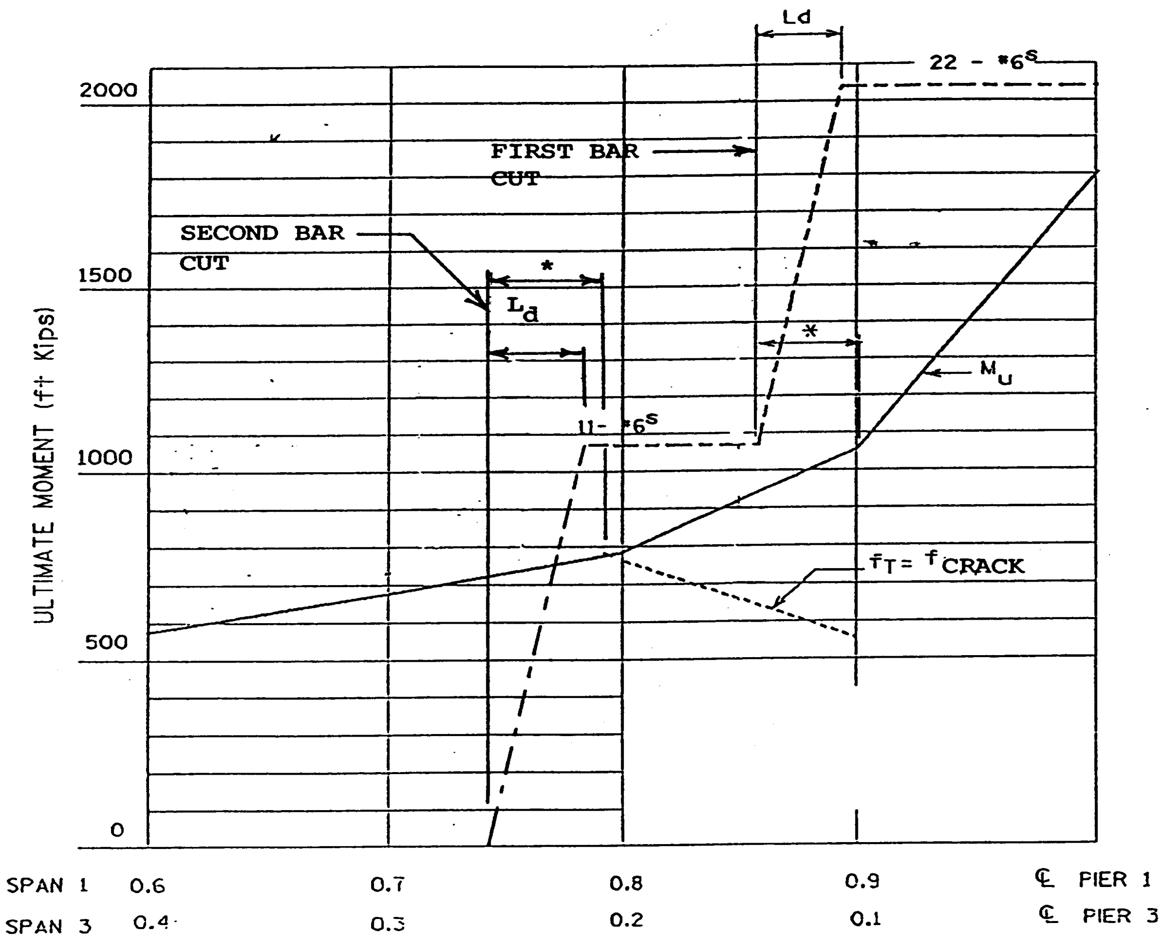
Prest. f_t = (-930/560) (1-(13.26 x 24.73)/223.9) x 0.76 = 0.586 ksi

Interior DL f_t = 1.795 ksi

Prestress + DL f_t = 0.586 -1.795 = -1.209 Ksi

Negative Stress Req'd. for cracking = 1.209 + 0.580 = 1.789 ksi
Negative Moment Req'd. for cracking = 1.789 x 5070/12 = 756 ft-kips
0.9 Span e = -11.10

Prest. f_t = -1.661 (1-(11.10 x 24.73)/223.9) x 0.760 = 0.285 ksi
Int. DL f_t = -1.006 Ksi
f_{CR} = 1.006 - .285 + .580 = 1.301 ksi
M_{CR} = 1.301 x 5070/12 = 550 ft-kips



'*' = EFFECTIVE DEPTH OF MEMBER

**The second bar cut was extended past the modulus of rupture point a distance equal to "EFFECTIVE DEPTH OF MEMBER".

ULTIMATE NEGATIVE MOMENT DIAGRAM

Note that for demonstration purposes, only the interior girders of Span 1 & 3 have been considered.

Extension length (See Chapter 9, Table 9.4) to determine tension splice length

Precompression
$p = A_s bd = 9.68/22 \times 49.63 = .0089 < 0.015$ OK

Camber and Deflection

Prestress Camber (Based on .75 f_s' minus ES loss) P = 930 x (202.5 - 18.3/202.5 = 846 Kips

L = 80.5' or 966" E = 3.5 x 10^3 ksi I = 125390. in^4
e' = eccentricity of all strands at end of girder
= 20.27 - (8 x 2 + 8 x 4 + 6 x 6 + 32 x 4)/30 = 13.20"
e" = center eccentricity - end eccentricity = (20.27 - 4.13) - 13.20 = 2.94"

Prestress Camber = $\dfrac{Pe'L^2}{8EI} + \dfrac{23\,PE''L^2}{216EI}$
= $(966)^2$ x [(846 x 13.2)27 + 23 (846 x 2.94)] (216 x 3.5 x 10^3 x 125390) = 3.53"
DL $\Delta = 5wL^4/384\,EI$ = 5 x (.583/12) x $(966)^4$/(384 x 3.5 x 10^3 x 125390.) = 1.26"

Prestress Camber = 3.53 - 1.26 = 2.27" Say: A = 2-1/4"

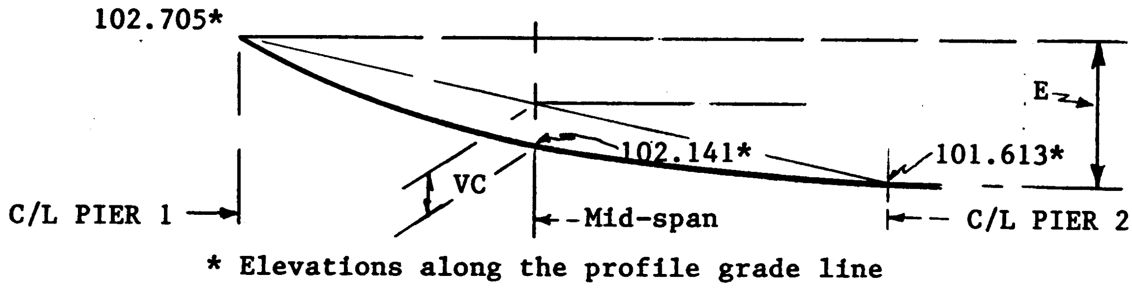
Dead Load Deflection

DL = 5 x (.956/12) x $(966)^4$/(384 x 5.5 x 10^3 x 125390) + 0.364 x $(966)^3$ /(48 x 5.5 10^3 x 125390) = 1.31 + 0.01 = 1.32" Say: B = 1-1/4"

Residual Camber = 2-1/4 - (1-1/4) = 1.0" Say: C = 1.0"

Deck Forming
Span 1 is on a straight grade, therefore VC = 0.

Span 2:



E/2 = (102.705 - 101.613)/2 = 0.546
101.613
102.159 - 102.141 = 0.018' Say VC = 1/4"

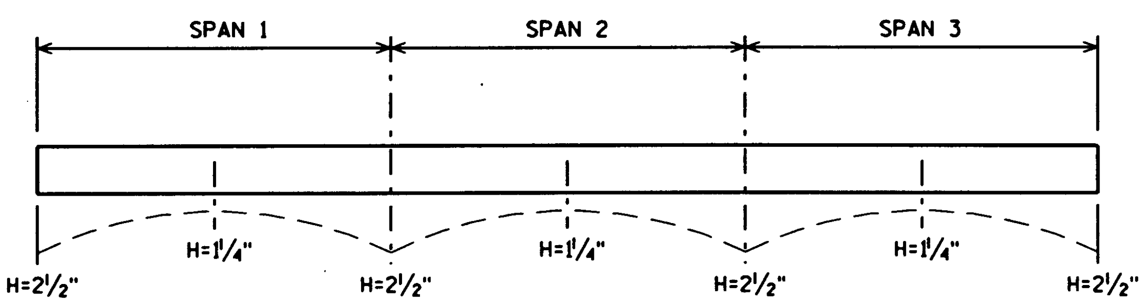
Span 3: VC = 1/4"

Span 1: Try 1-1/2" Haunch at Midspan
End Haunch: H_{END} = 1.0 - 0 + 1-1/2 = 2-1/2"

Span 2: Try Min 1-1/4" Haunch at Midspan
(H_{LT} = 2-1/2") + H_{RT} = 2(1.0 + 1/4 + 1-1/4) = 5.0"
Right Haunch: H_{RT} = 5.0" - (2-1/2") = 2-1/2"

Span 3: Try Min 1-1/4" Haunch at Midspan
2-1/2" + H_{RT} = 2 (1.0 + 1/4 + 1-1/4) = 5.0"

Right Haunch: H_{RT} = 2-1/2"



DECK FORMING DIAGRAM

B. 'Box Section Beam'

This design example is for a simple span pretensioned box "multi-beam" structure having a 2" minimum concrete overlay and is designed for a 20 pound per square foot future wearing surface.

Structure Preliminary Data

Span Lengths: 44 ft.-0 in. (simple span)
Live Load: HS20
Skew: 0°
(A-1) Abutments at both ends.
Concrete (prestressed box girder): f'c = 5,000 p.s.i.
Prestressed strands (low relaxation) - 1/2" f with Ultimate Tensile Strength = 270,000 p.s.i.
Bar Steel Reinforcement, Grade 60: fy = 60,000 p.s.i.
Concrete wt. = 150#/ft^3. (for box girder and overlay)
Parapet wt. = 45#/ft. (each). - Type "F"
Roadway Width: 28 ft. minimum.

Based on preliminary data (Sect. 19.3 (4)A) of this chapter and Table (19.2), select a Pretensioned Box Section having a depth of 1 ft.-9 in. by 4 ft.-0 in. wide, as shown on Bridge Manual Standard 19.15. Actual total deck width provided is 7 sections (4 ft.-0 in. wide) plus 6 (1 1/2" wide joints) = 28 ft.-9 in..

Live Load Distribution (AASHTO 3.23.4)

Load Fraction = S/D

S = 4.0 ft. (width of precast member).

$D = (5.75 - 0.5N_L)+0.7N_L\,(1 - 0.2C)^2$

$= (5.75 - 0.5(2))+0.7(2)(1 - 0.2(.653))^2$ ° 5.808

where $C = K\left(\frac{W}{L}\right) = 1.0\left(\frac{28.750\text{ ft.}}{44.0\text{ ft.}}\right) = 0.653$, and N_L ° 2(# of Lanes)

Load Fraction = 4.0/5.808 = 0.689

Live Load + Impact (AASHTO 3.8)

Impact = 50/(44+125) = 0.296

Live Load Moments

Inter. gir. LLM = (520.9/2)(0.689)(1.296) = 232.6 ft-k (See AASHTO Appendix "A")

Exter. gir. LLM = (0.1)(16)(1.296)(44) = 91.2 ft-k (See Sect. 19.3(4)(C) of Chapter19)

Dead Loads

Interior Box Section having a 2" minimum concrete
overlay thickness = (2"+2.625")/2 = 2.313" avg.

Box Section: (594/144)(0.15) -- = 0.619 k/ft

Conc. Overlay: (2.313/12)(4)(1)(0.15) ---------------------------------- = 0.116 k/ft

Future Wearing Surface: (4)(0.02) -- = 0.080 k/ft

Joint Grout: (1.5/12)[(21+2.313)/12](0.15) ------------------------------ = 0.036 k/ft

Total Inter. gir. DL = 0.851 k/ft

Dead Load Moments

Inter. (total) DLM = $(0.851)(44)^2/8$ = 205.9 ft-k

Inter. (gir. wgt. only) DLM = $(0.619)(44)^2/8$ = 149.8 ft-k

Using a rail dead load of 0.045 k/ft, results in a Total Exterior girder D.L.M. = 212.5 ft-k. Note an exterior box girder section design will not be provided in this example. However, the exterior girder must be analyzed for the railing DL, conc. overlay, F.W.S. and joint grout DL and also the LLM on the exterior girder section. The exterior girder shall not have less load carrying capacity then the interior girder.

<u>Section Properties</u> (21" x 48" wide) Box Girder

$A = 594$ in^2 $\qquad$ $I = 32{,}942$ in^4

$S_b = 3{,}137$ in^3 $\qquad$ $r^2 = I/A = 55.458$ in^2

<u>Interior Box Girder Section DL + LL Stresses</u> (@ Bottom)

f(total DL) = (205.9)(12)/(3,137) = 0.788 ksi

f(gir. DL only) = (149.8)(12)/(3,137) = 0.573 ksi

f(LL+I) = (232.6)(12)/(3,137) = 0.890 ksi

Final tensile stress allowed (@ bottom)= 6 $\sqrt{5000}$ = 0.424 ksi

Required stress due to prestress force at bottom of section to counteract service loads

= (f_p)

$f_p = 0.788 + 0.890 - 0.424 = 1.254$ ksi

<u>Allowable Stresses</u> (AASHTO 19.15.2)

f'ci º compressive strength of concrete @ time of initial prestress

use f'ci (Min.) º 4,000 p.s.i.

f'ci (Max.) º 4,500 p.s.i.

Initial top @ transfer: 7.5 $\sqrt{f'_{ci}}$ = 7.5 $\sqrt{4000}$ = + 474 psi

7.5 $\sqrt{4500}$ = + 503 psi

Final top @ service loads: -0.40 f'_c = -0.4(5000) = - 2000 psi

Initial bottom @ transfer: -.6f'_{ci} = -.6(4000) = - 2400 psi

= -.6(4500) = - 2700 psi

Final bottom @ service loads: 6 $\sqrt{f'_c}$ = 6 $\sqrt{5000}$ = + 424 psi

Estimate # of Strands Req'd, % Losses, and Prestress Forces

Strand eccentricity e = -10.5 + 2 = - 8.5" (from c.g. of sect. to CL of strands)

Referring to previous designs, assume 13 = # (strands) and approx. losses are,

Total Losses = 6000 + 6621 + 10,143 + 3531 = 26,295 psi

Est. % Total Losses = [(26,295)/(0.75)(270,000)](100) = 13.0%

Max Force per strand = (0.75)(270)(0.1531) = 31.003 kips

Est. Force per strand after losses = (31.003)(1. - 0.130) = 26,970 kips

Now estimate total prestress force (P), required to provide stress @ bottom º (fp),

Value of (fp) is on page 46.

$$P \text{ req'd} = \frac{-f_p A}{\left(1 + \frac{eY}{r^2}\right)} = \frac{(-1.254)(594)}{\left(1 + \frac{(-8.5)(-10.5)}{55.458}\right)} = -285.47 \text{ kips}$$

Est. of # strands req'd = 285.47/26.970 = 10.58

For initial trial analyze using 13 strands.

Total Initial Prestress Force (before losses) = Pi

Pi = (13)(31.003) = 403.0 kips

Elastic Shortening Loss (AASHTO 9.16.2.1.2)

Assume loss equals 3%

$$f_B = -\frac{Pi}{A}\left(1 + \frac{eY}{r^2}\right) = -\frac{403.0}{594}[1 + (-8.5)(-10.5/55.458)]$$

$$= 1.770 \text{ ksi}$$

with 3% Loss = 1.717 ksi

$$f_T = \frac{-403.0}{594}[1 - (+8.5)(10.5/55.458)]$$

$= +0.413$ ksi

$= +0.401$ ksi with 3% loss

or referring to AASHTO 9.16.2.1.2 to approximate force remaining in strands

immediately after transfer.

$Pi - P_{ES} = (13)(31.003)(0.69)/0.75 = 370.8$ kips (remaining prestress force)

from which $f_B = -1.628$ ksi and $f_T = +0.380$ ksi

As a comparison to previously computed values, these values are slightly lower.

Computing stresses at the 5/10 point for ES loss as (Prestress - ES + Beam DL):

$f_{Bi} = -1.628+0.573 = -1.051$ ksi (@ 5/10 point) stress @ transfer (bott.).

$f_{Ti} = 0.380 - 0.573 =$ 0.193 ksi (@ 5/10 point) stress @ transfer (top).

Computing stress @ c.g. of strands @ 5/10 point.

$f_{cir} = 1.051 - [(1.051 -0.193)(2/21)] = 0.969$ ksi

$ES = 7f_{cir} = (7)(0.969) = 6.785$ ksi

or referring to AASHTO 9.16.2.1.2

$ES = \frac{E_s}{E_{ci}} f_{cir} = 7.303\ f_{cir} = 7.077$ ksi

where $E_s = 28x10^6$ and $E_{ci} = 33w^{1.5}\ \sqrt{f'_{ci}}$

ES loss = (6.785)/(202.5) = 0.034 or 3.4%

Creep of Concrete (AASHTO 9.16.2.1.3)

Future wearing surface is not included, computing stress due to conc. Overlay DLM.

$$f_B = -f_T = \frac{\left[\frac{(0.036 + 0.116)(44)^2}{8}\right](12)}{3{,}137} = 0.141 \text{ ksi}$$

f_{cds} = 0.141-[(0.141+0.141)(2/17)] = 0.108 ksi

$CR_c = 12\ f_{cir} - 7f_{cds}$

= (12)(0.969)-(7)(0.108) = 10.872 ksi

Shrinkage (AASHTO 9.16.2.1.1)

SH = 17,000-150(72) = 6,200 psi = 6.2 ksi based on an average relative humidity of 72%.

Creep of Prestressing Steel (AASHTO 9.16.2.1.4)

CR_s = 5,000 - 0.10ES - 0.05 ($SH+CR_c$)

= 5,000 - 0.10(6785)-0.05(6,200+10,872)

= 3,468 psi = 3.468 ksi

Total Prestress Losses

Summation of Losses = 6,785 + 6,200 + 10,872 + 3,468

= 27,325 psi

% Losses = [27.325/202.5] (100) = 13.49%

Final Stresses after losses(@ CL span)

f_B = (-1.770)(0.865) + 0.788 + 0.890

= 0.147 ksi < 6 $\sqrt{f'_c}$ O.K.

f_T = (0.413)(0.865) - 0.788 - 0.890

= -1.321 ksi < 0.4 f'_c O.K.

13 Strands are adequate

Initial Stresses @ Transfer (@ Girder Ends)

f_B = -1.770(0.966) = - 1.710 ksi < .6 f'ci

f_T = +0.413(0.966) = + 0.399 ksi < 7.5 $\sqrt{f'ci}$

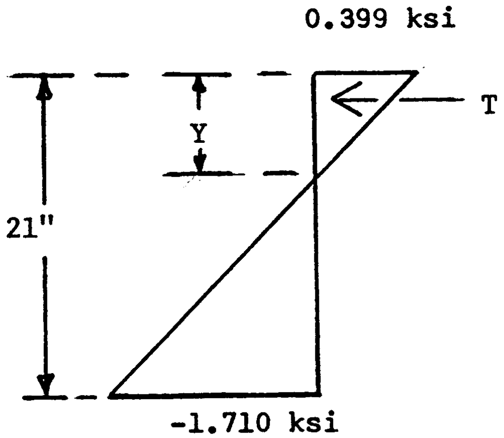
Req'd f'_{ci} = 1.710/0.6 = 2.850 ksi (Comp)

Req'd f'_{ci} = $\frac{(399)^2}{7.5^2} \times 10^{-3}$ = 2.830 ksi (Tens)

Both f'ci are less than minimum strength @ transfer of 4,000 p.s.i.

Use f'_{ci} = 4,000 psi.

<u>Non-Prestressed Reinforcement</u> (Req'd near top of girder) (AASHTO 9.15.2.1)



Y = (399)(21)/(399+1710) = 3.973"

T = (0.399)(0.5)(48)(3.973)

= 38.05 kips

A_s req'd = 38.05/24.0 = 1.59 in^2

A_s prov'd = (7)(0.31) = 2.17 in^2

Use: <u>7-#5 bars</u> full length of Beam

<u>Shear</u> (AASHTO 9.20)

Shear Strength Provided by Concrete (Vci or Vcw).

<u>Calculate V_{ci}</u>:

f_{pe} = (1.770)(0.865) = 1.531 ksi = 1531 psi (@ gird. ends)

f_d = 0 (@ gird. ends) Y_t = 10.5"

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f'_c} + f_{pe} - f_d\right)$$

$$= \frac{32{,}942}{(10.5)(12{,}000)}(424 + 1531)$$

= 511.1 ft-k

(b'd) = (2)(5)(19) = 190 in^2

$$V_d = \frac{[44]}{2}(0.851 \text{ k/ft.}) = 18.722 \text{ k (@ gird. ends)}$$

$V_{LL+I} = \dfrac{[56.7]}{2}$ (0.689)(1.296) = 25.315 kips (See AASHTO Appendix "A")

V_i = (2.17)(25.315) = 54.93 kips/ beam (@ gird. ends).

Mmax = (2.17)(232.6) = 504.7 ft-k (@ CL span) -- conserv. approach

$V_{ci} = 0.6\sqrt{f'_c}\,(b'd) + V_d + V_i\,M_{cr}/M_{Max}$

$= 0.6\sqrt{5000}\,(190) + 18{,}722 + (54{,}930)(511.1)/504.7$

V_{ci} = 82.4 kips per beam

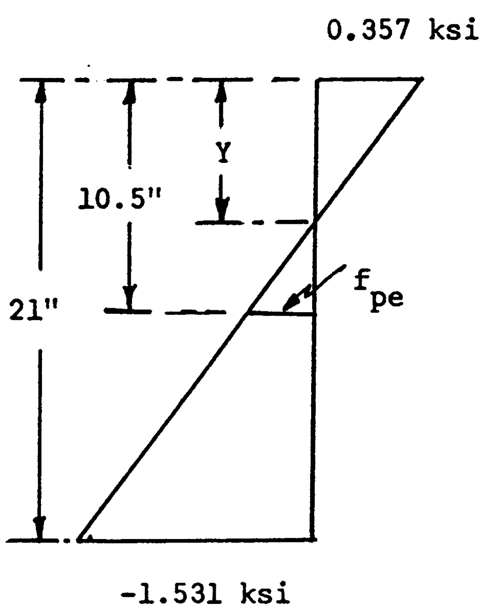
Calculate V_{cw}:

Note % prestress losses is 13.5%. Top and bottom stresses (@ gird. ends) after losses

are:

f_T = (0.413)(0.865) = 0.357 ksi

f_B = (-1.770)(0.865) = -1.531 ksi



$Y = [0.357/(0.357+1.531)]\,21 = 3.971"$

$f_{pc} = \dfrac{(10.5 - 3.97)}{(21 - 3.97)}\,(1.531)$

$= -0.587$ ksi

$V_p = 0$ (prestress strands not draped)

$0.8h = 0.8(21) = 16.8" < 19" = d$

$V_{cw} = (3.5\sqrt{f'_c} + .3f_{pc})(b'd) + V_p$

$= (3.5\sqrt{5000} + .3(.587))(190)$

$V_{cw} = (0.247 + 0.176)(190) = 80.4$ kips

Use: $V_c = V_{cw}$ = 80.4 kips per beam

| V_u/ϕ = [1.3 V_{DL} + 2.17 V_{LL+I}] 0.90

= [1.3(18.772) + 2.17(25.315)]/0.9 = 88.2 kips

| $V_s = \dfrac{V_u}{\phi} - V_c$

= 88.2 - 80.4 = 7.8 kips, note V_s is less than 4 $\sqrt{f'c}$ b'd.

s $\leq$ to 24" or 0.75h = 0.75(21) = 15.75" say: 15"

(Min.) $A_v = \dfrac{50b's}{f_{sy}} = \dfrac{(50)(10)(15)}{60{,}000} = 0.125\ in^2$

(Req'd) $A_v = \dfrac{(7.8^k)(15")(1000)}{(60{,}000)(19")} = 0.103\ in^2$.

Use #4 bar Stirrups @ 15" ctrs.

Ultimate Beam Capacity (AASHTO 9.17)

Interior gird. Mu = (1.3)(205.9) + (2.17)(232.6) = 772.4 ft-k (Design moment)

p* = (13)(.1531)/(19 x 48) = 0.00218; $\delta * \equiv 0.28$; $\beta_1 \equiv 0.80$

$f_{su}^* = f's\,[1 - \dfrac{0.35p^*f's}{f'_c}] = 270\,[1 - \dfrac{0.35(0.00218)(270)}{5.00}]$

= 258.9 ksi

$\phi Mn = \phi[A_s^* f_{su}^* d\,(1 - \dfrac{0.6p^* f_{su}^*}{f'_c})]$

$= (1.0)\,[(13)(.1531)(258.9)(19/12)(1 - \dfrac{0.6(0.00218)(258.9)}{5.00})]$

= 760.6 ft-k < 772.4 ft-k No Good.

Cracking Moment Check (AASHTO 9.18.2) - Min. reinf. requirement.

(Stress due to Prestress, after losses) f_B = (-1.770)(0.865) = -1.535 ksi (@ bottom)

$f_r = 7.5\ \sqrt{5000} \times 10^{-3}$ = 0.530 ksi (modulus of rupture)

f_{cr} = 0.530 + 1.535 = 2.065 ksi (stress req'd. for cracking) @ bottom

M^*_{cr} = (2.065)(3137) = 6478 in-kips

$$\text{Ratio of } \frac{øM_n}{M^*_{cr}} = \frac{(760.6)(12)}{6478} = 1.41 > 1.2 \text{ O.K.}$$

Final Note: 13 strands were used in showing the procedure for designing a prestressed box girder section. Because 13 strands were not adequate to meet Ultimate Moment Capacity requirements; in practice the designer would repeat the previous calculations with a greater number of strands until all requirements are met.

Camber and Deflection

Overall length (of gird.) = 43.833 ft. = 526"

M = (13)(31.003)(0.966)(8.5) = 3309 in.-kips.

Prestress Δ =526)^2(3309)/(8x3.5x10^3)(32,942)

= 0.993" Say: 1.0" (upward)

Beam Δ_{DL} = (5)(0.619/12)(526)^4/(384)(3.5x10^3)(32,942)

= 0.446" Say: 0.45" (downward)

Conc. overlay Δ_{DL} = (5)(0.116/12)(526)^4/(384)(3.5x10^3)(32,942)

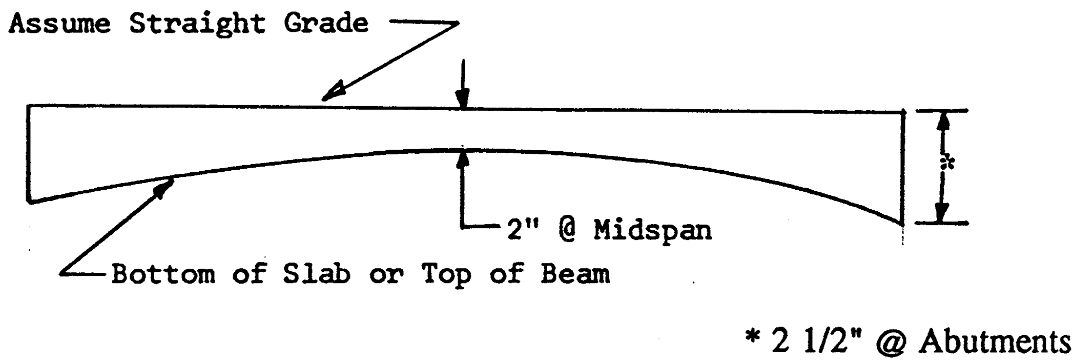
= 0.084" Say: 0.08" (downward)

Prestress Camber = 1.00 -0.45 = 0.55 Say: 5/8" (upward)

Dead Load Deflection = 0.08" Say: 1/8" (downward)

Residual Camber = 0.45" Say: 1/2" (upward)

Floor Forming



FLOOR THICKNESS DIAGRAM

See AASHTO Section 9 "Prestressed Concrete" and Standard 19.15 in the Bridge Design Manual for other requirements.

19.5 FIELD ADJUSTMENTS OF PRESTRESS FORCE

When strands are tensioned in open or unheated areas during cold weather they are subject to loss due to change in temperature. This loss can be compensated for by noting the change in temperature of the strands between tensioning and initial set of the concrete. For purposes of uniformity the strand temperature at initial concrete set is taken as 27°C (80°F).

Minor changes in temperature have negligible effects on the prestress force, therefore only at strand temperatures of 10°C (50°F) and lower are increases in the tensioning force made.

Since "plan" prestress forces are based on 75% of the ultimate for low relaxation strands it is necessary to utilize the AASHTO allowable of temporarily overstressing up to 80% to provide for the losses associated with fabrication.

The following example outlines these losses and shows the elongation computations which are used in conjunction with jack pressure gages to control the tensioning of the strands.

Computation for Field Adjustment of Prestress Force

Known:

22 - 1/2", 7 wire low relaxation strands $A = 0.1531$ in^2

P_i = 710.0 Kips (Initial prestress force from plan)

T_1 = 40°F. (Air temperature at strand tensioning)

T_2 = 80°F. (Concrete temperature of initial set)

L = 300' = 3600" (Distance from anchorage to reference point)

L_1 = 240' = 2880" (Length of cast segment)

E = 29000 ksi (Modulus of elasticity, sample tested from each spool)

C = .0000065 (Coef. of thermal expansion, per degree F.)

COMPUTE:

Force per strand = P = 710.2/22 = 32.3 Kips

$$DL_1 = \frac{PL}{AE} = \frac{32.3 \times 3600}{0.1531 \times 29{,}000} = 26.1''$$

Initial Load of 1.5 Kips to set the strands

$$DL_2 = \frac{1.5 \times 3600}{0.1531 \times 29000} = 1.22''$$

DL_3 = Slippage in Strand Anchors = 0.45" (Past Experience)

DL_4 = Movement in Anchoring Abutments = 0.25" (Same)

$DL_5 = C \times L_1 \times (T_2 - T_1) = 0.0000065 \times 2880 \times 40 = 0.75''$

$P_{Loss} = DL_5 \times A \times E/L = 0.749 \times 0.1531 \times 29000/3600 = 0.9$ Kips

Total Prestress Force = $P + P_{Loss} = 32.3 + 0.9 = 33.2$ Kips

Total Elongation = $DL_1 + DL_3 + DL_4 + DL_5 = 27.55''$

Elongation After Initial Load = 27.55 - 1.22 = 26.33"

19.6 PRESTRESS GIRDER STRAND PATTERNS

The basic criteria used in establishing the "standard strand patterns" is arranging the strands so that the c.g. of the patterns is as close to the bottom of the girder as possible.

Although this approach yields the most efficient capacity, it occasionally is modified to meet some of the stress conditions along the girder. These conditions are:

1) Maximum initial compressive stress in bottom fiber at transfer is $0.6f'_{ci}$.

2) Maximum initial tensile stress in top fiber at transfer is $7.5\sqrt{f'_{ci'}}$

In order to compute these values the dead load stresses of the girder are combined with the prestressing stresses. Average span lengths are assumed for each strand pattern with the length increasing as the number of strands increase. Due to this assumption certain strand patterns result in stresses that are too high when used on short spans with a wide girder spacing.

(3) The number of draped strands, which lie in the center two columns of strands, must be adequate to fulfill either:

A) Zero tension in the top fiber.

B) And a maximum of $0.4\ f'_c$ compression in the bottom fiber.

Also the amount of drape is controlled by the girder size and the 2" clearance from center of strand to top of girder.

(4) The minimum and maximum amount of reinforcement together with strand clearance and cover requirements of AASHTO are also used.

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